

GEOTECHNICAL PROPERTIES OF LIME STABILISED LOESS,

PORT HILLS, CANTERBURY

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ABSTRACT

The loess deposits of the Port Hills, Canterbury, are subject to erosion by natural processes. Urban development of the Port Hills has encountered difficulties due to loess erosion, and in some instances, contributed to erosion. Lime stabilisation is one remedial method that can be used to reduce erosion problems. This necessitates investigation of the changes to soil properties and strength gains that occur to the loess with the addition of lime.

An erodible loess colluvium, from a subdivision with a history of erosion problems, was stabilised with 1%, 2.5%, 5%, 7.5%, and 10% hydrated lime, and cured under various laboratory conditions. Pinhole erodibility, uniaxial swelling strain, Atterberg limits, grainsize analysis, Proctor compaction testing and a slaking test were used to determine the improvements to soil properties with the addition of lime. Unconfined compressive strength was tested to determine strength gains of lime stabilised soil.

The addition of 1% lime to the soil produced a non-erodible, non-dispersive material. Slaking and swelling were minimised with the addition of higher percentages of lime (5%-7.5% respectively). The effective grainsize of the soil was increased on the addition of lime, and plasticity was increased with the addition of up to 5% lime. Optimum moisture content increases, and dry density decreases with increasing amounts of lime.

Strength gains of the lime stabilised loess, varied from 3 - 14 times the strength of the untreated soil depending on the curing method. Strength gains were greatest for air dried samples, although the untreated soil cured in the same manner had a higher dry strength than the lime stabilised soil. Strength gains are optimised at and above 7.5% lime with significant strength losses recorded between 2% and 5% lime. Strain deformation is reduced with the addition of lime, and the modulus of deformation is increased significantly indicating that lime stabilised loess acts as a brittle material on deformation.

The addition of lime to loess in low percentages (1%), has

the effect of producing a non-erodible, non-dispersive material that resists erosion. However, it would appear that to achieve maximum strength of lime stabilised loess, 7.5% lime or more must be added to the soil.

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CHAPTER ONE

INTRODUCTION

1.1. AIMS AND SCOPE OF STUDY

Lime stabilisation has become one method used on the Port Hills in an attempt to control erosion problems associated with the loessial soil deposits. Although the use of lime to stabilize soils is a well established practice in other countries, it is a more recent practice on the Port Hills. Therefore there is a need to further investigate the effect of lime on these soils.

This project was undertaken with the aims of:

- 1) defining the erosion problems associated with loessial soils by setting up an erosion model typically found on the Port Hills;
- 2) to identify and characterise the erosive properties of loessial soil and investigate how these properties are effected by the addition of lime as a soil stabiliser;
- 3) providing statistical data on some of the strength properties of lime stabilised loess;
- 4) providing information on aspects of lime stabilisation for the local authorities that administer the development of the Port Hills.

The erosion model is defined from a proposed subdivision on the lower slopes of the Port Hills. The laboratory study is concentrated on the strength increases that occur when lime is added to the soil as a stabilising agent, but also covers other engineering properties that are affected by the addition of lime to the soil. These include;

- a) a decrease in the dispersive nature and erodibility potential of the soil,
- b) a decrease in shrink / swell potential,
- c) a decrease in the plasticity index,
- d) changes to grainsize,
- e) reduced slaking potential.

The effects of curing times and methods are also considered.

1.2. LOCATION

The Port Hills are the north-western flanks of the eroded Lyttelton volcano that is part of the extinct volcanic complex that makes up Banks Peninsula. The Port Hills separate the city of Christchurch from the port of Lyttelton (see fig. 1.1). A large percentage of these hills are mantled with loess soil deposits. These deposits vary in thickness and nature from site to site.

An increasing amount of residential development is taking place on the lower slopes of the Port Hills, as the city of Christchurch expands. Westmoreland is one such subdivision and has a history of instability problems associated with the erosion of loessial soils. Soil sampled from this subdivision is used in the laboratory section of this thesis (see fig. 1.1 for location). One site from the recently proposed subdivision of Whaka Terrace on the lower slopes of the Port Hills was mapped (see fig. 1.1 for location) and a typical erosion model for the Port Hills defined from this site.

1.3. GEOLOGICAL SETTING

1.3.1 Bedrock geology

Banks Peninsula consists of the eroded and partly drowned calderas of the Lyttelton and Akaroa volcanoes. These volcanic cones were constructed during the late Miocene and early Pliocene times (approximately 12-6 million years ago), and consist of basaltic and andesitic lava flows, agglomerates and ash beds (Bell 1978).

Older "basement" rocks of the Torlesse group (approximately 240 million years old) composed of greywacke and argillite strata are exposed at Gebbies Pass. Cretaceous volcanics (McQueen's Valley volcanics, approximately 95 million years old) also occur in the Gebbies Pass area. The Charteris Bay sandstone (approximately 65 million years old) indicates a period of marine deposition between the Cretaceous volcanics and the Miocene volcanics that formed the Banks Peninsula volcanic complex. Fig. 1.2. shows the detailed bedrock geology of Banks Peninsula.

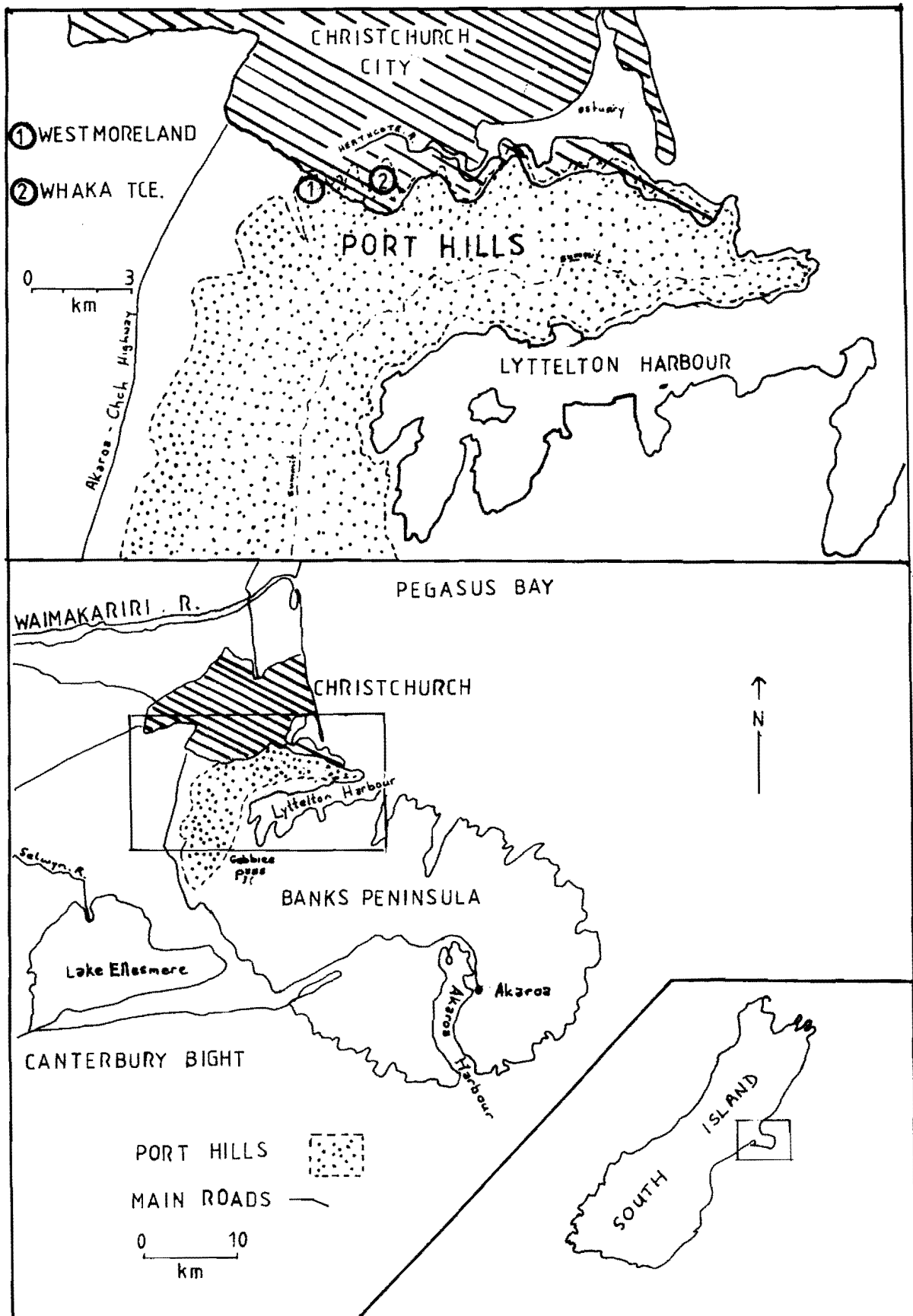


Fig. 1.1. Location map. Banks Peninsula, the Port Hills, and the two sites, Westmoreland and Whaka Tce., studied in this thesis

1.3.2 Geomorphic development

The Lyttleton - Akaroa volcanic cones are estimated to have reached a maximum height of 1500-1800 metres above present sea level (Bell 1978). It is evident from the channelled lava flows that erosion was active as the cones were being constructed. As volcanic activity ceased the cones were dissected by radial drainage systems flowing from their flanks and the earlier established drainage channels were further incised. The eroded central caldera areas of both cones were breached (the exact time is not known) and the present day Lyttleton and Akaroa Harbours formed.

The Banks Peninsula volcanic complex developed initially as an island. However the deposition of gravel outwash fans from the Southern Alps during the Pleistocene connected the north-western part of the island to the mainland forming the Canterbury Plains and Banks Peninsula (Liggett and Gregg 1965). During this time aeolian material was deposited on the flanks of the volcanoes and subsequently eroded and redeposited.

1.3.3 Regolith deposits of Banks Peninsula

(i) Volcanic colluvium

Volcanic colluvium consists of weakly to moderately weathered volcanic rock fragments set in a matrix of clay loam or silty clay loam. The matrix is composed of highly to completely weathered fines derived from volcanic bedrock.

Volcanic colluvium occurs mainly on moderately steep mid-backslopes below bedrock outcrops. Thicknesses are extremely variable due to the very active erosion conditions at these sites. Volcanic colluvium usually overlies weakly weathered volcanic basement.

(ii) Mixed colluvium

Mixed colluvium consists of loess colluvium mixed with weathered volcanic material. The ratio of loess colluvium to volcanic colluvium in the mixture ranges from 10% to 90% and as a result the morphology is highly variable.

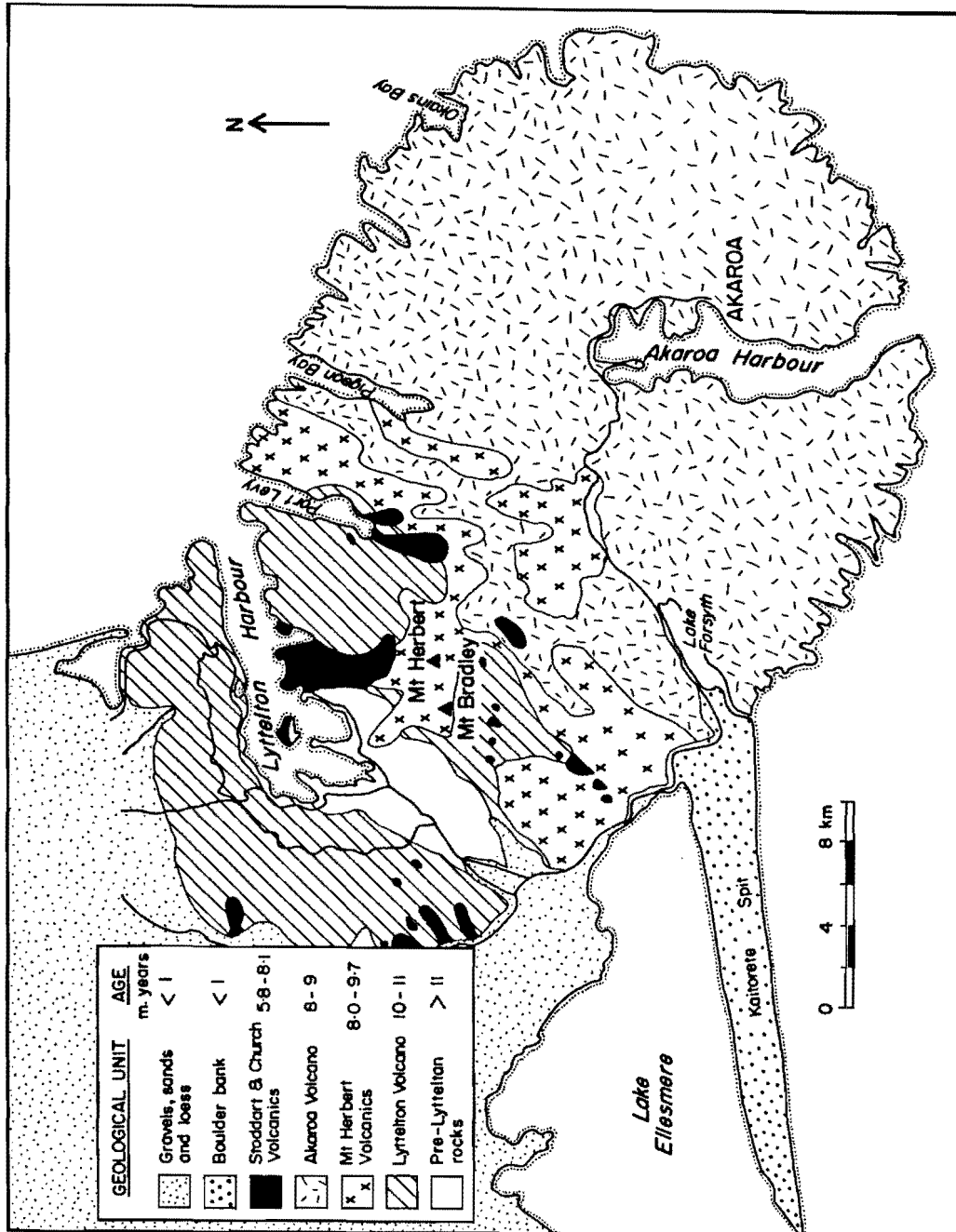


Fig. 1.2. Simplified geological map of Banks Peninsula. (Weaver et al., 1985).

Measured thicknesses of mixed colluvium vary between 0.4 and 3 metres. On the Port Hills, the mixed colluvium occurs on backslopes and upper footslopes, below outcrops of volcanic rock. In natural landscapes on the Port Hills, regoliths of mixed colluvium are those most prone to failure by mass movement (Bell and Trangmar (in prep.)).

1.3.4 Loess deposits of Banks Peninsula

(i) Origin of Loess Deposits

During Pliestocene glaciations of the southern alps, large amounts of fine grained material was produced by glacial grinding action. This material was eventually transported by the dominant north-westerly wind regime and deposited as an airfall blanket of loess over the eroded flanks of Banks Peninsula (Bell 1978, Bell and Trangmar (in prep)). Raeside (1964) suggests that areas of the continental shelf exposed at low stands of sea level during the Pliestocene, also contributed fine grained material which was deposited by easterly and southerly winds.

Griffiths (1973, 1974) suggests that much of the loess cover on Banks Peninsula was eroded during episodes of deposition, due to freeze-thaw induced mass movement, with some of the material accumulating on the lower slopes (accounting for the observed thickening of the loess blanket at lower elevations). Paleosols and other layers (reworked loess) between different loess members substantiates this. A distinction must therefore be made between in-situ (primary airfall) loess and reworked loess (loess colluvium).

(ii) Nature, distribution and thickness of loess

Griffiths (1973) recognises two distinct types of loess on Banks Peninsula, as shown in Fig.1.3.

- 1) Birdlings Flat Loess: is a coarse, calcareous (fine sandy loam) loess which is thickest on the north-north west facing slopes at the lower elevations (maximum measured thickness 16m). A colluvial layer separates two major loess layers. In the uneroded sections a mottled fragipan with bleached vertical veins occurs in the upper layer. Hard calcareous concretions are often present and the lime may cement the finer soil particles into sand- size composites.

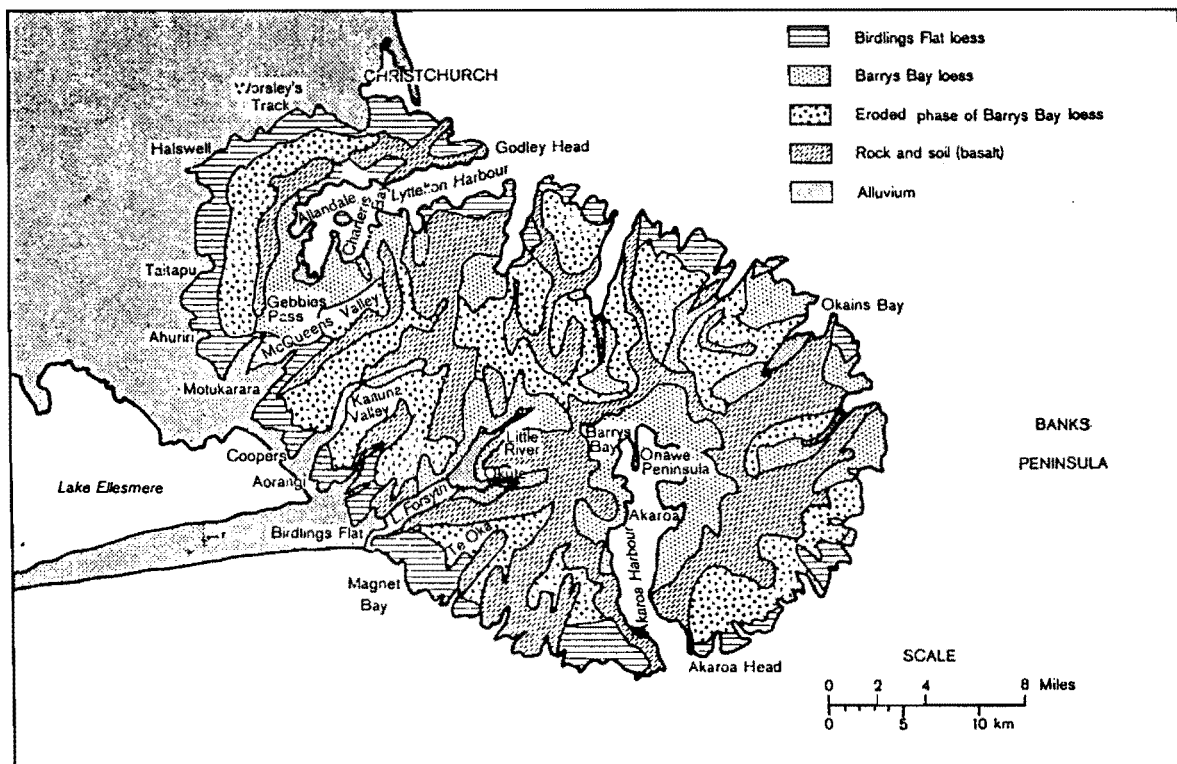


Fig. 1.3 Distribution of loess on Banks Peninsula (from Griffiths, 1973).

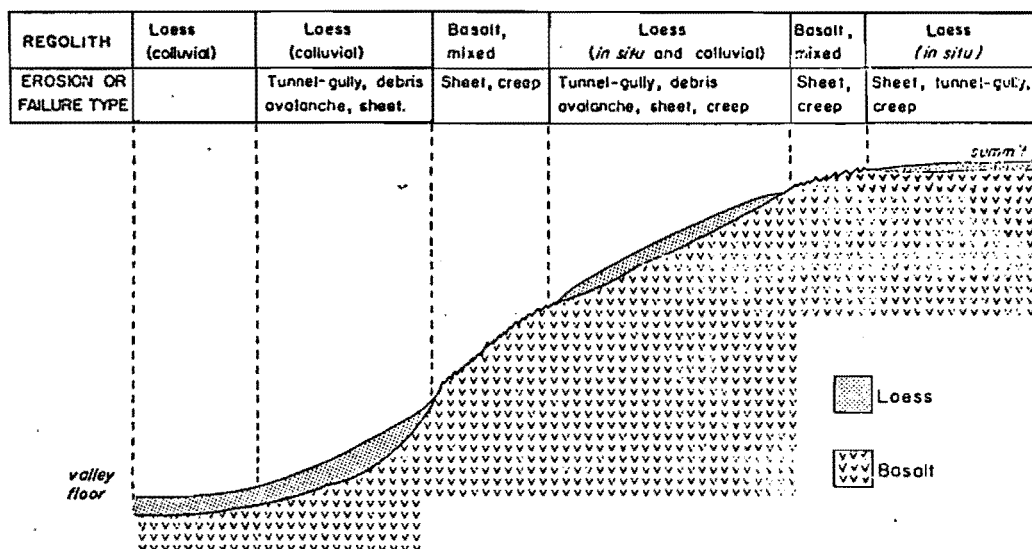


Fig. 1.4. Typical profile showing present day loess distribution on the Port Hills and the associated erosion processes. (Bell, 1978).

- 2) Barry's Bay loess: is a non calcareous loess of finer grain size (silt loam - fine sandy loam texture) than the Birdlings Flat Loess. It is found at the head of inlets, upper slopes and summit regions of Banks Peninsula. Four major loess layers are recognised at the type area. Each layer is mottled with distinct vertical and horizontal grey veins. The maximum thickness of loess recorded, is about 12m.

Each facies is composed predominantly of quartzo-feldspathic minerals, with minor accessory minerals (e.g epidote, zircon and tourmaline) and some secondary clay minerals such as illites and vermiculites (Bell & Trangmar (in prep) after Raeside 1964 and Griffiths 1973).

Loess colluvium includes all loessial materials transported downslope since initial deposition. It is principally composed of quartzo-feldspathic silts and fine sands but contains up to 10% volcanic rock fragments (Bell & Trangmar (in prep.)). The volcanic component increases with proximity to bedrock and the loess colluvium often grades into a mixed loess/volcanic colluvium (See section 1.3.3).

Loess colluvium occurs mostly on the lower slopes of Banks Peninsula. The thickness of the loess colluvium varies from 0.5 to 20m+ and may overlie basement volcanics, older colluvium or insitu loess (Bell and Trangmar (in prep.)). Figure 1.4 shows the distribution of these deposits on a typical Port Hills slope section.

1.4. METHODS

1.4.1. Field Methods

Field work included mapping of the site at the Whaka Terrace subdivision. Three seismic surveys were carried out in an attempt to define the bedrock geology and 17 auger holes drilled and logged to aid geological interpretations. A well exposed soil profile was logged in a collapsed tunnel gully. Tube and bag samples were taken

from this profile and geotechnical properties tested. An erosion model was based on this information.

1.4.2. Laboratory Methods

The analysis is basically a laboratory study of the properties of lime stabilised loess. A comparison of strength and other geotechnical properties is made between the untreated soil and soils treated with 1%, 2.5%, 5%, 7.5%, and 10% lime addition (by dry weight). Hydrated lime, Ca(OH)_2 , was used throughout the study as it is a safer chemical to handle and store than other forms of lime.

The effects of different curing methods and various curing periods on the strength, slaking potential, erodibility and swelling potential of stabilised and unstabilised soil are also considered.

CHAPTER TWO

REVIEW OF LOESS EROSION AND ASSOCIATED ENGINEERING PROBLEMS

2.1. INTRODUCTION

This chapter reviews the erosion problems associated with loess deposits of Banks Peninsula and the Port Hills, and summarises some of the problems encountered, and remedial measures used, during development on the Port Hills. A tunnel gully erosion model from the Whaka Terrace subdivision attempts to characterise some of the soil properties responsible for soil erosion and tunnel gully formation. The development history of Westmoreland subdivision provides an example of erosion problems encountered when working with erosive soils.

2.2. EROSION PROCESSES ASSOCIATED WITH LOESS DEPOSITS

Griffiths (1973,1974) concluded that natural erosion processes occurred during the episodic deposition of loess under the influence of the colder Pliocene climate regime. Although no longer under this regime, the erosion of the loessial blanket on Banks Peninsula is still occurring. These erosion processes were probably accelerated by fire induced removal of native forest in Pre-European times and by the logging of forest remnants by Europeans in the 19th Century (Bell & Trangmar in prep). Urban development and agricultural practises on Banks Peninsula in the last hundred years have also led to an increase in the rate of erosion of the loessial blanket.

Bell (1978) and has recognised five major types of erosion processes occurring on Banks Peninsula and proposed the following classification;

- 1) Rock and Debris Falls
- 2) Soil Creep
- 3) Slide - Avalanche - Flow mass movements
- 4) Sheet and Rill erosion
- 5) Tunnel - Gully erosion

2.2.1 Rock and Debris Falls

Although rock and debris falls may involve loess falling from a free face, this type of erosion process mainly occurs with toppling volcanic bedrock from out-crops or cliffs and will not be covered further in this study.

2.2.2 Soil Creep

Soil creep involves the slow (imperceptible) down slope movement of the weathered surface zone, in which temperature and water content fluctuate seasonally. Soil creep occurs in zones of high water content above a layer of low permeability, which may act as a poorly defined failure surface. Creep is indicated by terracettes, ripples, and mounds on the ground surface, developed parallel to the contour of the slope.

On Banks Peninsula, soil creep mostly occurs in in-situ loess and colluvial regoliths on the wetter south to east facing slopes that have a shady aspect. Slope angles vary between 20 and 38 degrees. Under high intensity rainstorm conditions soil creep often develops into rapid, shallow, slide failures.

2.2.3 Slide - Avalanche - Flow Mass Movements

This type of mass movement occurs on a clearly defined shear surface by rapid channelised "avalanching"; by flowage of a sediment - water mixture; or by a combination of both, and grades into mass transport by stream floods (Bell & Trangmar (in prep.)). As with soil creep, accumulation of water over a zone of low permeability defines a failure surface.

The majority of slope failures on Banks Peninsula occur in colluvial deposits, (few have been observed in insitu loess), on slopes steeper than 20 degrees with a shady (southerly-easterly) aspect. They are commonly associated with seepage zones of groundwater discharge where the soil is already saturated, and mass movement is triggered by high intensity rainstorms. Progressive failures may occur under prolonged rainfalls (Bell 1978).

2.2.4 Sheet and Rill Erosion

Sheet erosion is the downslope movement of soil particles by surface water flows and occurs on poorly vegetated slopes under high intensity rainfalls. Rill gullies develop as a result of channelling by water. Sheet and Rill erosion are of minor importance on Banks Peninsula but recently cultivated land and areas of poor vegetation cover are highly susceptible to this type of erosion (Hosking 1962). Figure 2.1. shows an example of sheet and rill erosion of Port Hills loess.

2.2.5 Tunnel Gully Erosion

Tunnel gully erosion is a major erosional problem on Banks Peninsula (and especially the Port Hills, due to urban development on the lower slopes). The mechanisms involved in the development of a tunnel gully are complex. The model given in Fig.2.2. (from Bell and Trangmar (in prep.) shows the processes involved and they are summarised below:

- 1) The depletion of vegetation cover which promotes soil dessication, and shrinkage cracks develop from the surface downwards;
- 2) Subsequent infiltration of water from either natural (surface & groundwater) or "artificial" sources;
- 3) Dispersion of the clay mineral fraction to form initial subsurface flow paths;
- 4) Tunnel enlarging by slaking;
- 5) Physical erosion of collapsed debris within the tunnel by intermittent water flows;
- 6) Progressive enlargement of tunnel network and ultimately collapse of the bridging soil mass to form open gullies.

The clay fraction in loess (normally not greater than 20%) plays a significant role in the formation of tunnel gullies. A high percentage of exchangeable sodium cations in the clay (dependent on the mineralogy of the clay particles) results in the deflocculation of clays by seepage waters and the progressive collapse of the weakly cohesive soil aggregates (dispersion). This plus the pressure exerted on the soil by entrapped air on saturation leads to the disruption of the solid particle skeleton (slaking). Miller (1971) concluded that seasonal shrinkage of the soil mass (again a function of clay



Fig. 2.1 Sheet and rill erosion, loess landfill, Reserve block, Whaka Tce.



Fig. 2.3 Mature tunnel gully erosion, Hillsborough Tce, Port Hills.

mineralogy) was of prime importance in allowing access of water, and that both dispersion and slaking were dominant processes involved in tunnel formation.

Tunnel gully erosion on Banks peninsula mainly occurs within Birdlings Flat loess or within loess colluvium derived from it. Hughes (1972) demonstrated a statistical preference for tunnel gully development on west to north-west facing slopes (these slopes are seasonally dry) of slope angles between 5 and 30 degrees. Most tunnel gullies are developed on lower slopes (below 250m asl) where loess and colluvium thicknesses are greatest. Figure 2.3 shows mature tunnel gully erosion on the Port Hills.

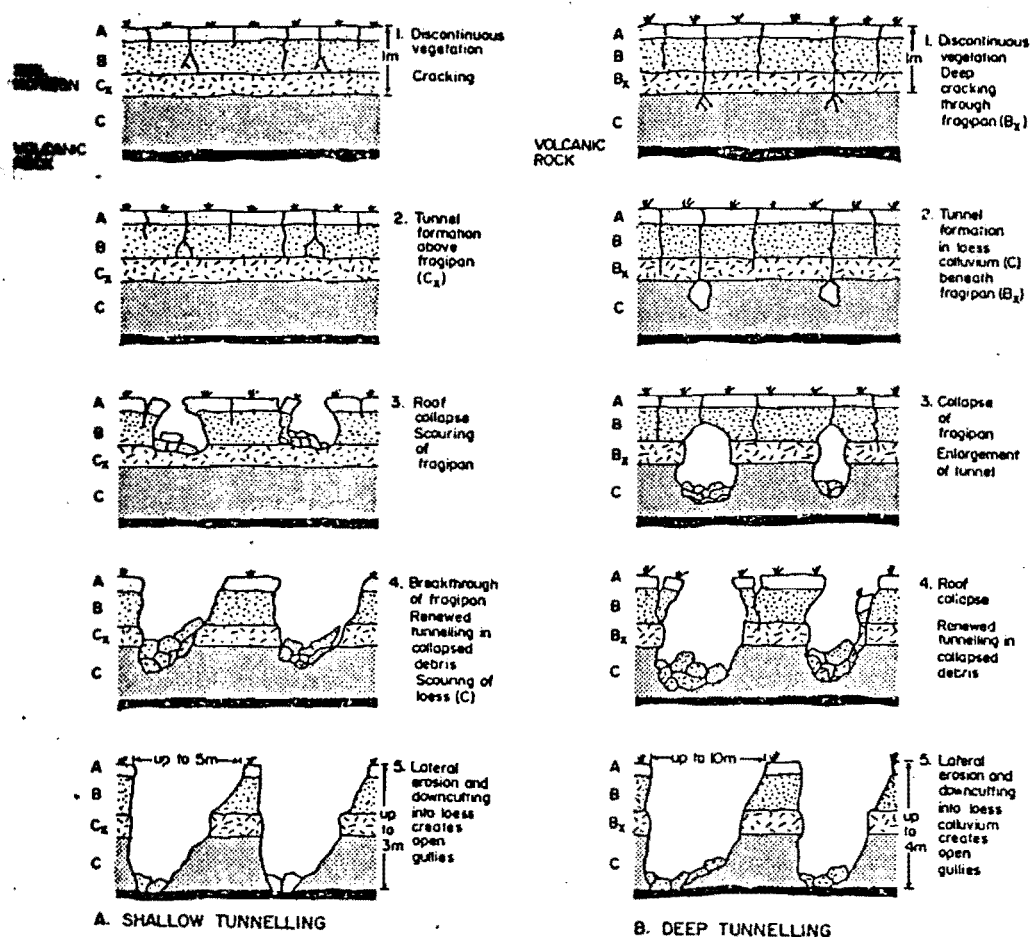


Fig. 2.2. Model for the formation of shallow and deep tunnel gully systems in loessial soils of the Port Hills. (Bell and Trangmar, in prep.).

2.3. WHAKA TERRACE TUNNEL GULLY EROSION MODEL

2.3.1 Introduction

A proposed subdivision on the lower slopes of the Port Hills provides a tunnel gully erosion model. The subdivision plan contains 14 house sites and a reserve block (see fig 2.4, map pocket). Roading and services are also to be provided.

Site investigation methods included 3 seismic refraction surveys, the drilling of hand auger holes and the engineering geological mapping of the site at a scale of 1:500. A soil profile was logged from a collapsed tunnel gully. Geotechnical properties of 35mm tube samples and bag samples taken from the logged section were tested in the laboratory.

2.3.2 Site description

The subdivision is to be developed on slopes of loess colluvium and irregular outcrops of volcanic bedrock. Slope angles vary from 10 to 38 degrees and have a westerly aspect. Major tunnel gullies are a prominent feature across the site. Thick gorse covers the lower regions of the site. Figure 2.5 provides a panoramic view of the site.

(i) Bedrock geology

Volcanic bedrock exposed at the site is a reddish grey slightly to moderately weathered basalt. Highly to completely weathered volcanic ash deposits were found locally in auger holes drilled on blocks 10 and 11 in the upper regions of the site. The bedrock is exposed along the south-eastern boundary of the site, on the existing track between blocks 2 and 12, and a bedrock knoll or high is exposed on block 12. Discontinuous outcrops also occur on the colluvium covered slopes of blocks 6 and 7 (see figs 2.4 and 2.5).

It appears, from the discontinuous and irregular nature of the exposed bedrock and from bedrock profiles interpreted from the seismic refraction surveys, that the volcanic rock occurs as benches, probably formed by differential weathering of the various flows and ash deposits.



Fig. 2.5. Panoramic view showing the site of the proposed subdivision at Whaka Tce (outlined). Loess deposits can be seen in the middle ground behind the existing house. Bedrock is also indicated.

(ii) Volcanic and mixed colluvium .

Volcanic and mixed loess/volcanic colluvium occurs locally over the site. Thicknesses of the volcanic and mixed colluvium vary from 10cm to 80cm.

(iii) Loess colluvium .

Loess colluvium has been deposited over the slope in varying thicknesses. In general, two types of loess are recognised;

- 1) A yellowish brown homogeneous clayey silt (ML) which is slightly weathered in parts but appears to be non-erodible and,
- 2) An olive-grey brown homogeneous clayey silt with some fine sand (ML) which appears to be erodible and dispersive.

More detailed descriptions are given in the auger hole logs and the soil profile (fig. 2.6, map pocket).

Seismic refraction surveys (see fig. 2.6, map pocket) indicate that there is a "paleo-depression" or "paleo-gully" between bedrock outcrops of block 7 and block 12 and that there is up to 7m of colluvium deposits in this depression. Information from auger holes indicates that bedrock may only be up to 3m below the surface.

(iv) Tunnel gully erosion .

Tunnel gully erosion is a prominent feature of the site occurring over most of the central part of the development (see fig. 2.4). Major collapsed tunnel gullies (> 2m width) occur in the central part of the site on blocks 2, 3, 4, 9, and 14, where loess colluvium deposits are greatest.

Shallow tunnel gullies, no more than 50cm in diameter and no more than 1m below the ground surface, occur on blocks 5, 8, 9, 12, and 15. Not all of these gullies have collapsed to the surface and the extent of them can only be estimated. On blocks 8 and 9, the presence of such features is indicated by small collapsed holes, contrasting vegetation cover, and uneven topography. Small tunnel gully exits (< 10cm in diameter) are exposed in the batter of the existing track on block 5, but no surface expression of these gullies is evident on the slope above.

2.3.3 Tunnel gully erosion model

A logged soil profile from one of the major collapsed gullies on the site defines a typical erosion profile, commonly associated with tunnel gully erosion on the Port Hills. Figure 2.7. shows the detailed log with soil descriptions and some basic geotechnical properties obtained from 35mm tube samples and bag samples.

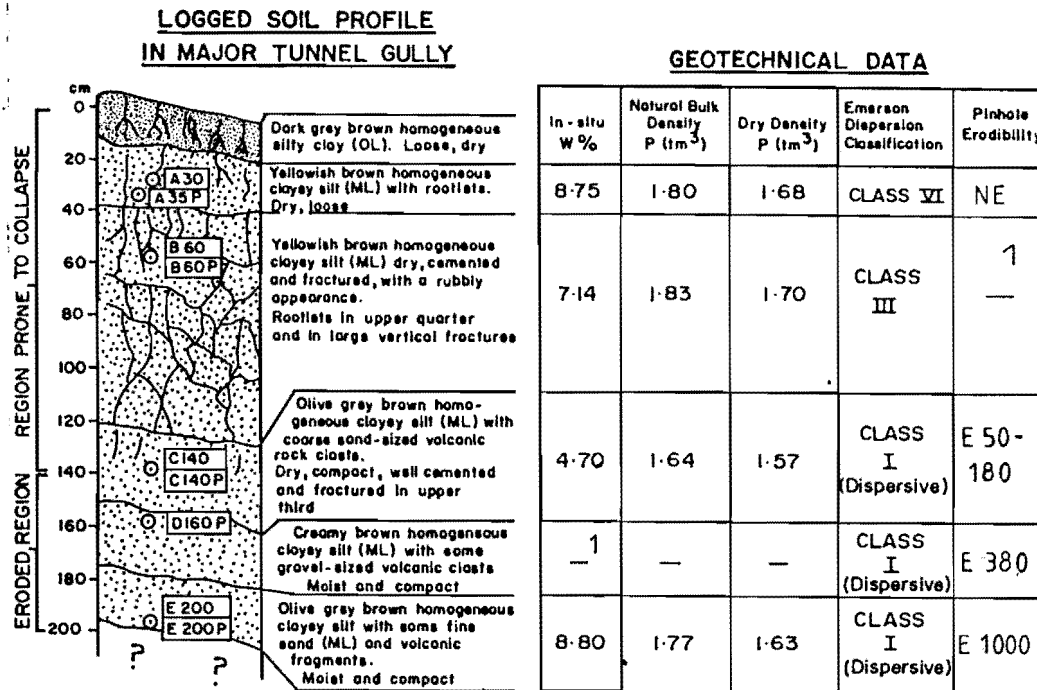


Fig. 2.7. Tunnel gully erosion profile, and table of tested geotechnical properties, Whaka Tce subdivision.

¹Sample destroyed before testing.

Six distinct layers are identified from the profile. The uppermost layer being the organic layer. Below this is a 20-25cm layer of loess that is non-dispersive and non-erodible. Between 40 and 120cm there is a well cemented and fractured layer with a rubbly appearance. This layer has the highest density of all the layers and although it is slightly dispersive (class III) it appears from field evidence to be non-erodible. This layer is comparable to the fragipan described by Evans (1977).

The region below 140cm has been eroded and a cavity approximately 50cm in diameter has formed. Except for a well cemented and fractured zone, just below the "fragipan" layer, the three lower layers (C, D, and E) all have dispersive and erosive properties,

making them susceptible to tunnel gully erosion. Erosion of the C, D, and E layers has removed the support of the upper three non-erodible layers, and they have subsequently collapsed into the void below. The source of water causing the erosion was not established, but it is thought to be derived from either a natural seepage zone (which are commonly found over the site) or discharge from sewer or stormwater services above the development site (see fig. 2.4).

2.4. ENGINEERING PROBLEMS CAUSED BY LOESS EROSION

2.4.1 General

The erosion problems discussed in 2.2. and 2.3 pose potential problems for urban development on the Port Hills. Tunnel gully erosion and slide - avalanche - flow mass movement are of the greatest concern.

Tunnel gully erosion may lead to the loss of foundation support, road collapse, or water and sediment discharge into storm water systems and basements. Slide - avalanche - flow mass movement can cause damage to surface structures or underground services. Soil creep may result in the cracking of foundations and paths and the tilting of fences, service pylons, and trees. Shallow underground services may also be disrupted. Sheet and rill erosion may cause sedimentation of drainage systems.

Poorly designed works in loessial soils on the Port Hills have at times initiated or contributed to erosion. The placement of fill material on a loessial slope without prior benching, creates a potential slide surface at the fill buried soil interface. Cut batters in loess material greater than 1m, may penetrate the fragipan layer, thereby creating a potential for tunnel gully formation in the underlying material. The installation of underground services by trenching and backfilling may create the potential for tunnel gully formation, as seepage waters tend to follow the path of the trench.

2.4.2 Erosion history of Westmoreland subdivision

(i) Introduction

Westmoreland subdivision (or Worsleys Spur subdivision) is an area of 125 hectares on the lower slopes of the Port Hills, which is being progressively developed for urban residential use. Since earthworks began in 1974, a number of the erosional problems outlined in section 2.2. have been encountered due to;

- 1) the nature of the loessial soils on which the development is taking place, and
- 2) poorly planned development and a lack of understanding of the problems that might be encountered.

(ii) History of earthworks

1974 . Earthworks commenced in February 1974 when two gullies were filled with large quantities of soil to a depth of 15m. By April 1974 silt was being washed from these gullies into the Cashmere stream and sub-surface erosion was occurring. The subdividers were required to construct a settling pond to ensure that silt settled before run-off was discharged into the stream.

1975 . The first stage was approved in 1975 and required the reshaping of the loess mantled slopes by cut and fill methods. During dry summer months the loess was being wind eroded and in later months bared surfaces were being eroded by water in the form of sheet, rill and gully erosion.

1976-1977 . Two excavations and two fill operations up to a depth of 2m were carried out. In February 1977 a large eroding tunnel gully (15m x 6m depth) was filled. These excavations were approved with the proviso that the work be carried out at appropriate times, the fill be keyed into undisturbed earth and compacted, and that the areas were grassed over and temporarily protected from erosion.

Storms occurred during June and July 1977 causing damage to new sewer and storm water services. Holes up to 5m in diameter had formed in places and tunnel gullies appeared on the lower slopes.

1978 . A detailed land survey was completed by the North Canterbury Catchment Board which provided a soil stability classification of the area and lead to a reclassification of land use of the area by the Paparua County Council.

1979-1982 . Work continued on the second and third stages of the subdivision. Work was approved with the developers having to adhere to controls and specifications for any further earthworks set down by the various local authorities (Bell, 1982b)

Work still continues on the final stages of the development in 1985.

2.5. SYNTHESIS

From the types of erosion processes that can occur on the loess covered slopes of the Port Hills, and from the erosion examples given, it can be seen that it is essential that the properties of the soils must be investigated before any works on loessial slopes takes place; and that such works are carefully designed. The use of a number of pre-cautionary construction methods (eg. the backfilling of services trenches with compacted lime stabilised soil, the recompacting and benching of filled ground, and the use of subsurface and surface drainage) can eliminate many erosional problems in the long term.

CHAPTER THREE

THE EFFECTS OF LIME ON BASIC SOIL PROPERTIES

3.1. INTRODUCTION

Until 1940, and the onset of World War 2, lime had not been used in construction practices to any large extent. Since then, and especially in the U.S.A., lime has been widely used to treat and stabilise clay - gravel aggregates, heavy clay soils, and as a suitable but less effective treatment of silty soils (Bell 1982). Lime stabilised soils have been used for road and airfield construction, canal linings, impervious cores of earth dams and other engineering projects.

The term "lime" can be applied to a number of substances. The more common of these are;

- 1) agricultural lime (CaCO_3),
- 2) burnt or quick lime (CaO), and
- 3) hydrated or slaked lime (Ca(OH)_2).

Hydrated lime is preferred for soil stabilisation because it does not undergo the volume expansion of quicklime on hydration (which readily occurs), and it is the safer chemical to handle and store. Agricultural lime is relatively unreactive with soil (Ferguson, 1982) and is not commonly used for stabilisation. Hydrated lime was used in this study, and the term "lime" refers to hydrated lime.

According to Bell, 1982, (after Winterkorn and Fang, 1975) the property changes that result from lime addition include;

- 1) a reduction in the plasticity index, and an increase in the plastic limit,
- 2) an increase in the optimum moisture content (omc) and a reduction in maximum dry density,
- 3) a decrease in clay sized particles due to flocculation,
- 4) a marked reduction in shrink / swell behaviour,
- 5) a marked increase in unconfined compressive strength and bearing capacity, and
- 6) the production of a water resistant material that minimises

the infiltration of gravity water.

This chapter deals with the property changes that occur to the silty loessial soils on the addition of lime, excluding the effects of lime on strength gains, which are dealt with in chapter 4.

3.2. PREVIOUS RESEARCH: THE MECHANISMS OF SOIL - LIME STABILISATION

3.2.1 Mechanisms of soil - lime stabilisation

Although lime has been widely used for soil stabilisation since 1940, the reactions of the lime with the soil were poorly understood. Research into the mechanisms of lime stabilisation is now, however, well documented, (Clare and Cruchley (1957), Eades and Grim (1960), Herrin and Mitchell (1961), Ingles (1962, 1968), Croft (1964), Diamond and Kinter (1965), and Stocker (1969, 1972), Ferguson (1982)). Reaction mechanisms of lime stabilisation have been attributed to one or more of the following;

- 1) clay mineral flocculation,
- 2) replacement of exchangeable cations by Ca^{2+} ions,
- 3) carbonation of atmospheric CO_2 to form calcium carbonate and,
- 4) the formation of the so-called pozzolanic hydrated calcium aluminates.

Diamond and Kinter (1965) discounted the importance of cation exchange, flocculation, and carbonation in the stabilising process. They recognised two distinct stages of reaction with lime; the first being the "rapid amelioration" (within hours) of water-sensitive properties such as plasticity, and the second the slow production (days to years) of cementitious materials responsible for strength gain. They argue that the initial reactions involve the physical adsorption of a mono-molecular layer of Ca^{2+} and 2OH^- ions onto clay mineral surfaces. (This in turn leads to flocculation of the particles and therefore the effects of flocculation are of some importance to the stabilising process). This is followed by the rapid formation of tetra calcium hydrate, and more slowly, calcium silicate hydrate (tobermorite gel) at the point contacts of the

flocculated clay particles (fig. 3.1). The longer term result of continuing reactions with sorbed lime is the formation of various calcium silicate hydrates and calcium aluminium hydrates, which crystallise in void spaces and act as a cementing product.

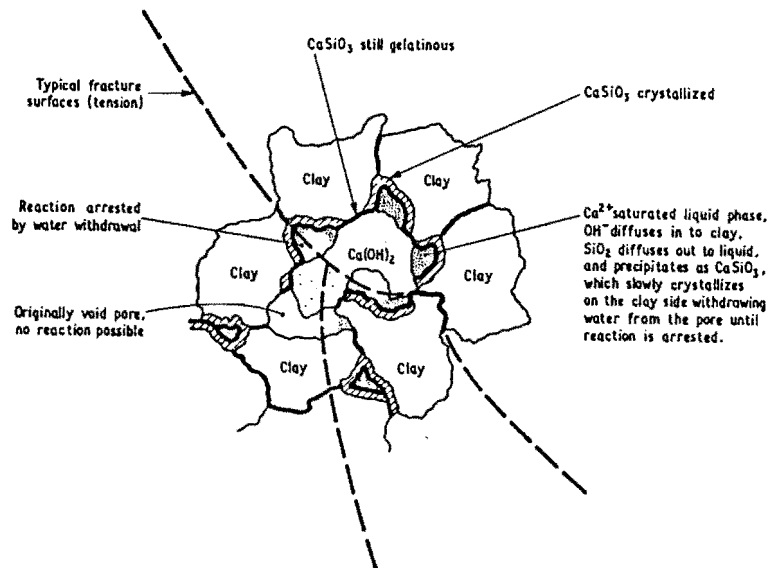


Fig 3.1. Diagram showing the mechanisms, site and nature of cementation in lime stabilised clayey soils as summarised by Ingles and Metcalf (1973).

3.2.2 Lime stabilisation of Port Hills loess

Evans (1978) conducted laboratory research into the stabilising effects of lime and phosphoric acid on loessial soils from two locations on the Port Hills. He concentrated on reducing the erosive and dispersive nature of the soils as well as comparing the relative effects of the two stabilisers.

Evans and Bell (1981) reported laboratory and field research into the effects of lime stabilisation of Port Hills loess. They attempted to optimise erosion resistance rather than to maximise strength gain and concluded that;

- 1) the addition of hydrated lime in concentrations as low

as 0.5% by weight of dried soil results in the formation of an erosion resistant material given adequate curing.

- 2) at 2% lime addition, potential swelling is minimised, and
- 3) that at about 5% lime addition, unimmersed unconfined compressive strength is maximised.

Bell (1982a, 1982b and 1981) has continued research on the use of lime as a stabilising agent for the erosive Port Hill soils concentrating on the field application techniques. MacNeill (1982) did a review of laboratory testing and field application techniques of lime stabilised loess used for the Paparua reservoir site on the Port Hills.

3.3. SOIL PROPERTY CHANGES OF LIME STABILISED LOESS

3.3.1 Scope of the testing program

The testing program was designed to investigate the soil property changes that occur with the addition of lime relevant to erosion processes (especially tunnel gully erosion) of Port Hills loess.

Although no one test can be used to determine whether a soil is suitable for lime stabilisation, the effects of lime on grain size, plasticity, optimum moisture content and dry density are parameters that should be investigated prior to the use of lime, to ascertain the response of the soil.

Pinhole erodibility, dispersion, swelling strain and slaking are relatively easy tests that can be used to determine physical improvements of the soil, with the addition of lime, that are relevant in reducing erosion potential of the loessial soils. The erosion processes have already been discussed in chapter 2.

3.3.2 Field sampling

A bulk sample of soil became available from a service trench in the Westmoreland subdivision in May 1984, and a quick characterisation of samples 35mm tube samples and bag samples was undertaken to assess the erosive and dispersive nature of the soil, and its suitability for

lime stabilisation. The soil was found to be erodible (E 180 - refer to section 3.3.7 and appendix 3) and highly dispersive (Emerson class I - refer to appendix 4). Grainsize analyses showed that there was sufficient clay sized material for lime stabilisation, although this is not a pre-requisite for lime stabilisation. Other soil properties of the field sample are included in table 3.1.

	in-situ moisture content(w %)	in-situ dry density t/m ³	SAND %	SILT %	CLAY %	Liquid Limit	Plastic Limit	Plasticity Index	Pinhole Erodibility	Emerson Dispersion Class	Swelling Strain ε _s %
FIELD SAMPLE W3	7	1.72	14	65	21	24	16	8	E 180	I	9.36

TABLE 3.1. GEOTECHNICAL PROPERTIES OF THE FIELD SAMPLE (W3) FROM WESTMORELAND SUBDIVISION

The untreated soil from Westmoreland is a loess colluvium and can be described using the soil material descriptions based on Bell and Pettinga (1984) as a;

dark yellowish brown, massive, slightly weathered, moist, firm, clayey silt with some fine to coarse sand and fine gravel clasts of moderately-highly weathered volcanic rock.

Mineralogy studies (microscope and XRD) show that the loess is composed of quartz and feldspar particles with minor fragments of volcanic minerals and volcanic rock. X-ray diffraction analysis indicates that the loess has smectite and mixed layered illite clay minerals, and possibly some vermiculite (see appendix 7).

3.3.3 Laboratory testing

Optimum moisture contents and maximum dry densities were obtained for the untreated soil, and soils treated with 1%, 2.5%, 5%, 7.5%, and 10% hydrated lime (expressed as weight percent of $(\text{Ca}(\text{OH})_2$ added by dry weight of soil), using standard compaction methods (see appendix 8). Batches of untreated and lime treated soil were then mixed at approximately optimum moisture content. These are given in figure 3.2. The lime - soil mixtures were stored for an initial

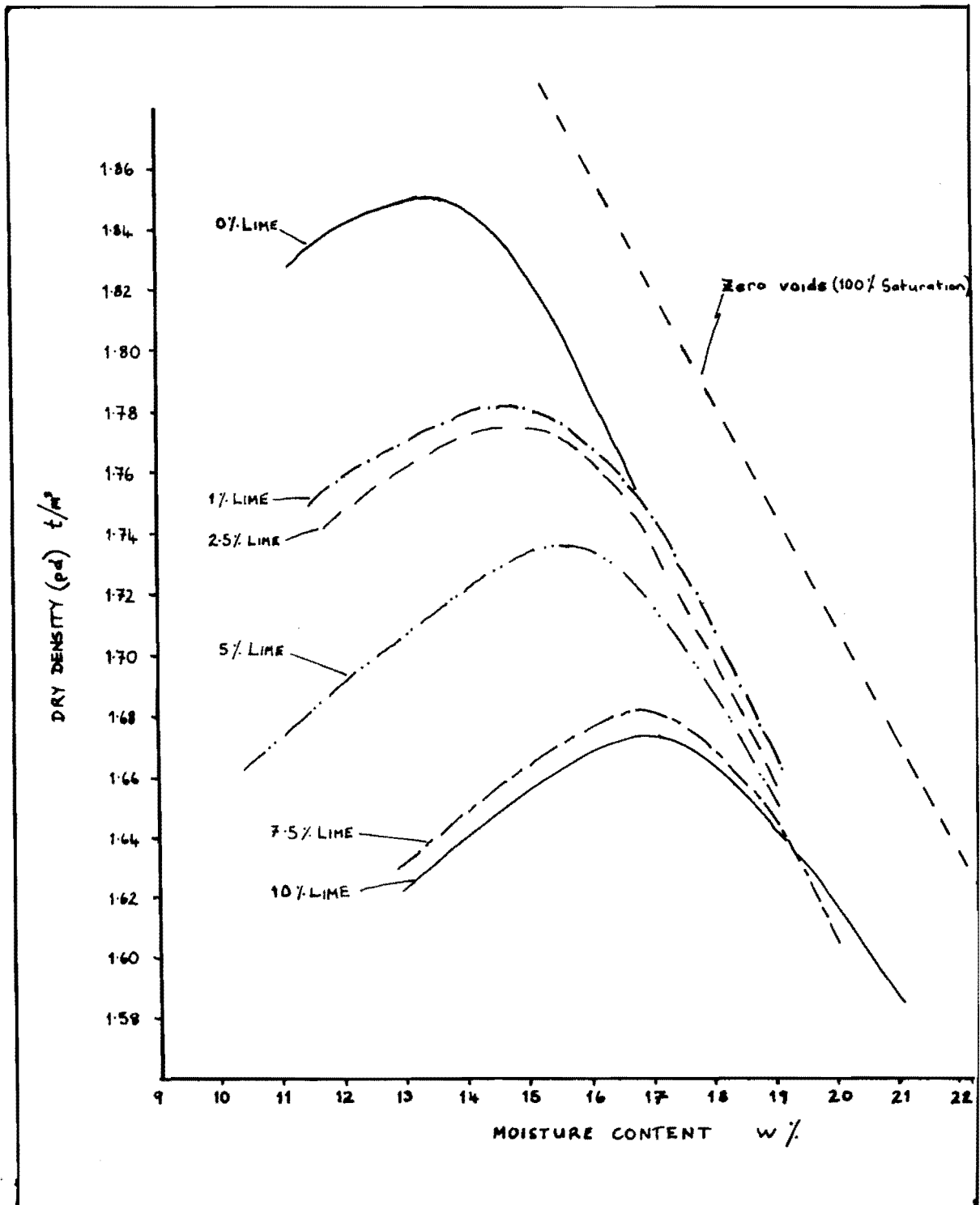
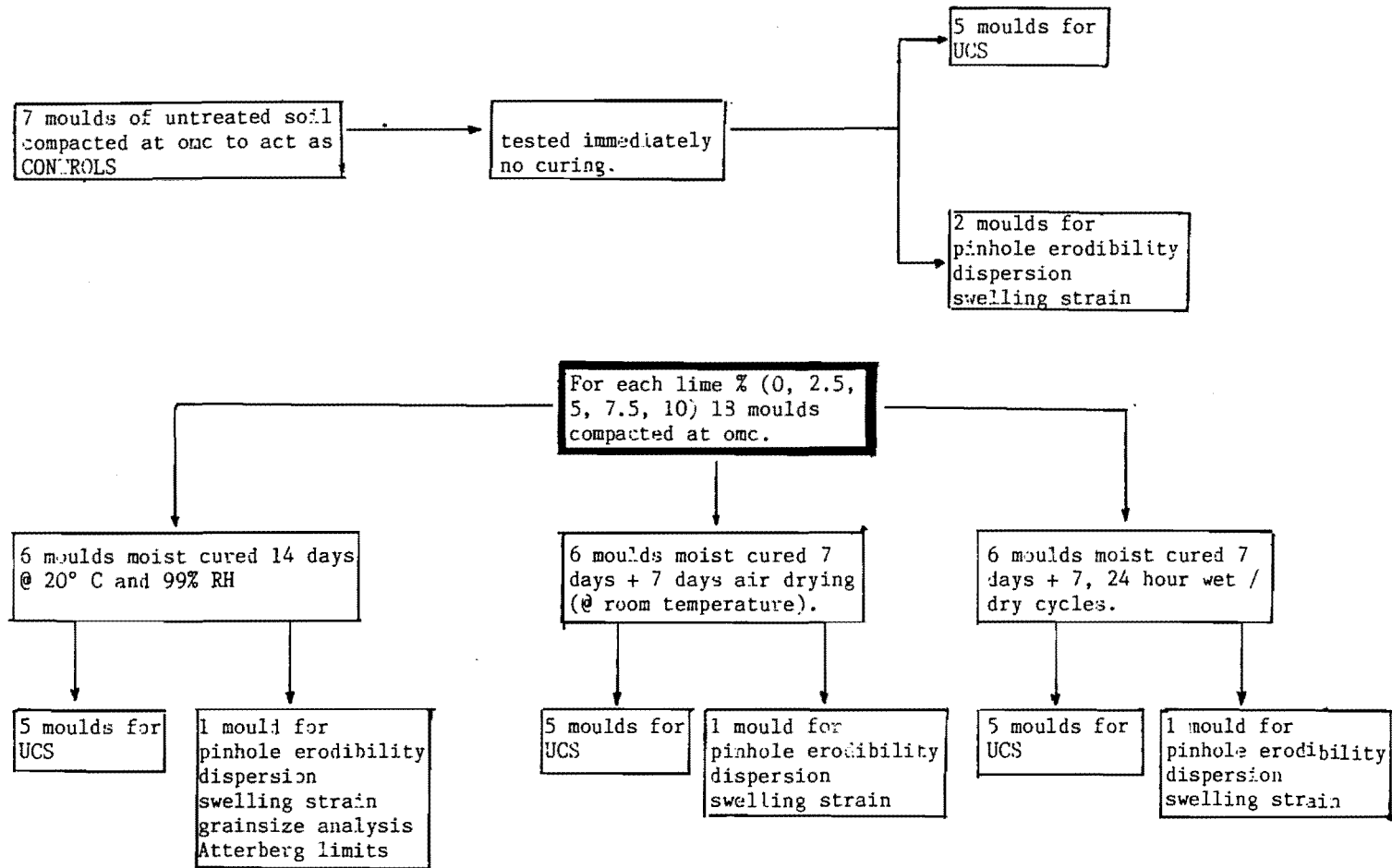


Fig. 3.2. Dry density - moisture content relationship for the untreated soil (0%) and for the soil treated with 1%, 2.5%, 5%, 7.5% and 10% lime.

Fig. 3.3. Simplified flow diagram of testing procedure.



curing period of 24 hours prior to compaction into Proctor moulds.

Eighteen moulds for each lime percentage and for the untreated soil were compacted using standard compaction methods. Six of these moulds were moist cured for 14 days (at 20°C and approximately 99% humidity), six were moist cured for 7 days followed by a period of air curing for 7 days, and six were moist cured for 7 days, then subjected to seven, 24 hour wetting and drying cycles. Different curing methods were used to determine their effects on property changes and strength gains. Curing methods are detailed in appendix 2.

One mould, for each lime percentage and each curing method, was used to obtain samples for pinhole erodibility, Emerson dispersion, and swelling strain testing. Samples for grainsize analyses and Atterberg limits were taken from the 14 day moist cured mould. The effects of slaking were determined on the moulds that underwent the wetting and drying cycles. The remaining moulds were used for testing unconfined compressive strength which is covered in chapter Four.

Figure 3.3 shows a simplified flow diagram of test procedures. Results are given in summary tables 3.2 and 3.3 and are discussed in the following sections.

3.3.4 Grainsize

Grainsize analyses (sieve and hydrometer method - see appendix 5) show that the untreated soil contains 21% clay, 65% silt and 14% sand sized particles or larger (by weight percent), (see table 3.2). The clay fraction is defined as $2\mu\text{m}$, the silt fraction, $2 - 60\mu\text{m}$, and the sand fraction, $60 - 2000\mu\text{m}$.

Previous studies (Brand and Schonenberg, 1959, Herrin and Mitchell, 1960, Brandl, 1981) have shown that the addition of lime increases the effective grainsize of the soil. Figure 3.4. show grainsize changes with the addition of increasing amounts of lime for the soil tested in this study.

The addition of 1% lime to the soil reduces the amount of clay sized particles slightly. The addition of lime to the soil has the effect of flocculating the clay minerals into aggregates by the

formation of ionic bonds (described in 3.2.1 - "rapid amelioration" effects). However, with 1% lime, there is an increase in the silt fraction and a corresponding reduction of the sand fraction. The addition of 2.5% lime has no effect on reducing the clay content further, but the trend of the sand and silt fraction is reversed by similar proportions, ie. the sand fraction is increased and the silt fraction is decreased.

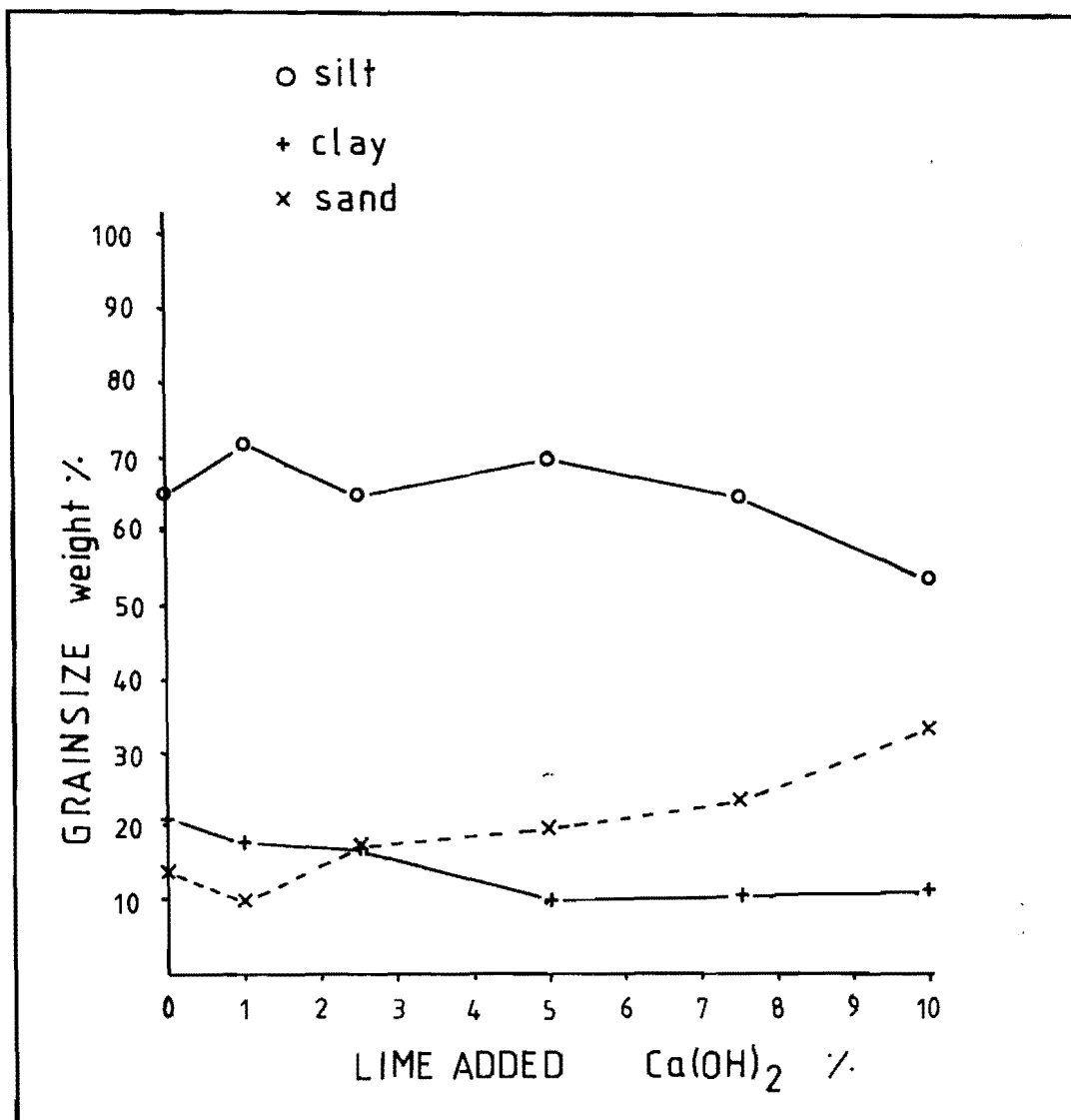


Fig. 3.4. The changes to grainsize distribution with the addition of lime to the soil. The data is based on the average of three samples.

It would appear that there is an initial reaction (lack of reaction or a delayed reaction) of the soil with lime that lowers the sand and clay fractions, but increases the silt fraction. It may be that on the initial addition of lime, existing soil aggregates are firstly broken down or rearranged in some way before the processes of clay particle flocculation are allowed to proceed.

Clare and Cruchley (1957), and Diamond and Kinter (1965), suggest that pozzolanic reaction products, in minute quantities, are produced immediately at clay particle contacts within the flocs, so that the flocs become meta-stable units; but that these early reaction products are not sufficient to bind flocs together. This may account for the irregularities in grainsize distribution with lower percentages of lime, in that the flocculated aggregates are unable to withstand the disaggregation process of the analysis (refer appendix 5).

With higher percentages of lime (5%+), the effects of clay mineral flocculation are more evident, with the flocculation of clay particles becoming more permanent, enabling them to withstand disaggregation. With the addition of 5% lime there is a significant reduction of the clay fraction, and a slight increase of the silt and sand fractions. The addition of more lime does not further reduce the clay fraction but reduces the silt fraction with a corresponding increase of the sand fraction. At this stage it appears that the lime is no longer flocculating clay minerals but binding flocs into larger aggregates.

The overall effect of the addition of higher percentages of lime (5%+), after 14 days moist curing higher, is to increase the relative grainsize of the soil, from a clayey silt to a silty clay.

3.3.5 Plasticity

Atterberg limits (see appendix 8) were determined for both the field sample and for the recompacted 14 days moist cured sample for each lime percentage. Results are given in tables 3.1 and 3.2 and in figure 3.5.

The plasticity indices for both the field sample and the

	Lime content %					
	0	1	2.5	5	7.5	10
Sand % 60-2000 μ m	14	10	17	20	24	34
Silt % 2-60 μ m	65	72	65	70	65	54
Clay % <2 μ m	21	18	18	10	11	12
Liquid limit LL	23	32	34	35	33	32
Plastic limit PL	16	18.5	21	22	22	NP
Plasticity Index PI	7	13.1	13	13	10	-
Soil Activity (A) ¹	0.33	0.75	0.72	1.3	0.91	-

1 Soil Activity as defined by Holtz and Kovacs, 1981 (after Skempton, 1953).

2 Denotes non-plastic.

Table 3.2 Grainsize, Atterberg limits, and Soil Activity for the untreated soil and lime treated soil.

untreated sample (0%, moist cured for 14 days) are similar, indicating that curing of untreated soils has no or little effect on the workability of the soil. The untreated soil can be classified as a low to non-plastic silt using Casagrandes classification and has a low soil activity typical of loessial soils (Grim, 1962 - see table 3.2).

In general, previous research (see below) has shown that the addition of lime increases the plastic limit of the soil with a corresponding decrease in the plasticity. The effect of lime on the liquid limit is less well defined. Herrin and Mitchell (1961) found that the liquid and plastic limit of soils with an initial low plasticity increased with the addition of lime, and resulted in an increase in the plasticity of the soil. Other researches (Clare and Cruchley 1957, Croft 1964, Eades and Grim 1960, Brandl 1981) have

found that soils with initial low plasticities are least reactive with lime and, that the plasticity of inactive soils is increased with the addition of lime.

The initial response of the lime is to flocculate clay particles (rapid amelioration effects as described by Diamond and Kinter, 1965 - see section 3.2) and attractive forces between soil particles increase, raising the liquid and plastic limits. However there is a point at which additional lime has little or no more affect on the plasticity (the lime fixation point as described by Diamond and Kinter, 1965).

Figure 3.5. shows that the plasticity index of the soil tested increases with the addition of 1% lime, due to a marked increase in the liquid limit and a slight increase in the plastic limit. This is presumably the result of rapid meta-stable flocculation as described in section 3.3.4. The liquid limit continues to increase slightly with the addition of up to 5% lime, and is followed by a decrease with any additional lime. Flocculation with higher percentages of lime becomes more permanent in nature with individual flocs being bound into stable aggregates.

The plastic limit of the soil also increases markedly with the initial addition of lime, but this increase levels off with the addition of 5% and 7.5% lime and the soil is rendered non-plastic on the addition of 10% lime. Correspondingly the plasticity index of the soil is initially sharply increased followed by a gradual decrease. However, at 7.5% lime the plasticity of the soil is higher than that of the untreated soil, although the relative plasticity is low ($PI=10$). Activity of the soil (see table 3.2) is also increased with the addition of lime, as found for other soils with low activity by Clare and Cruchley (1957) and Brandl (1981).

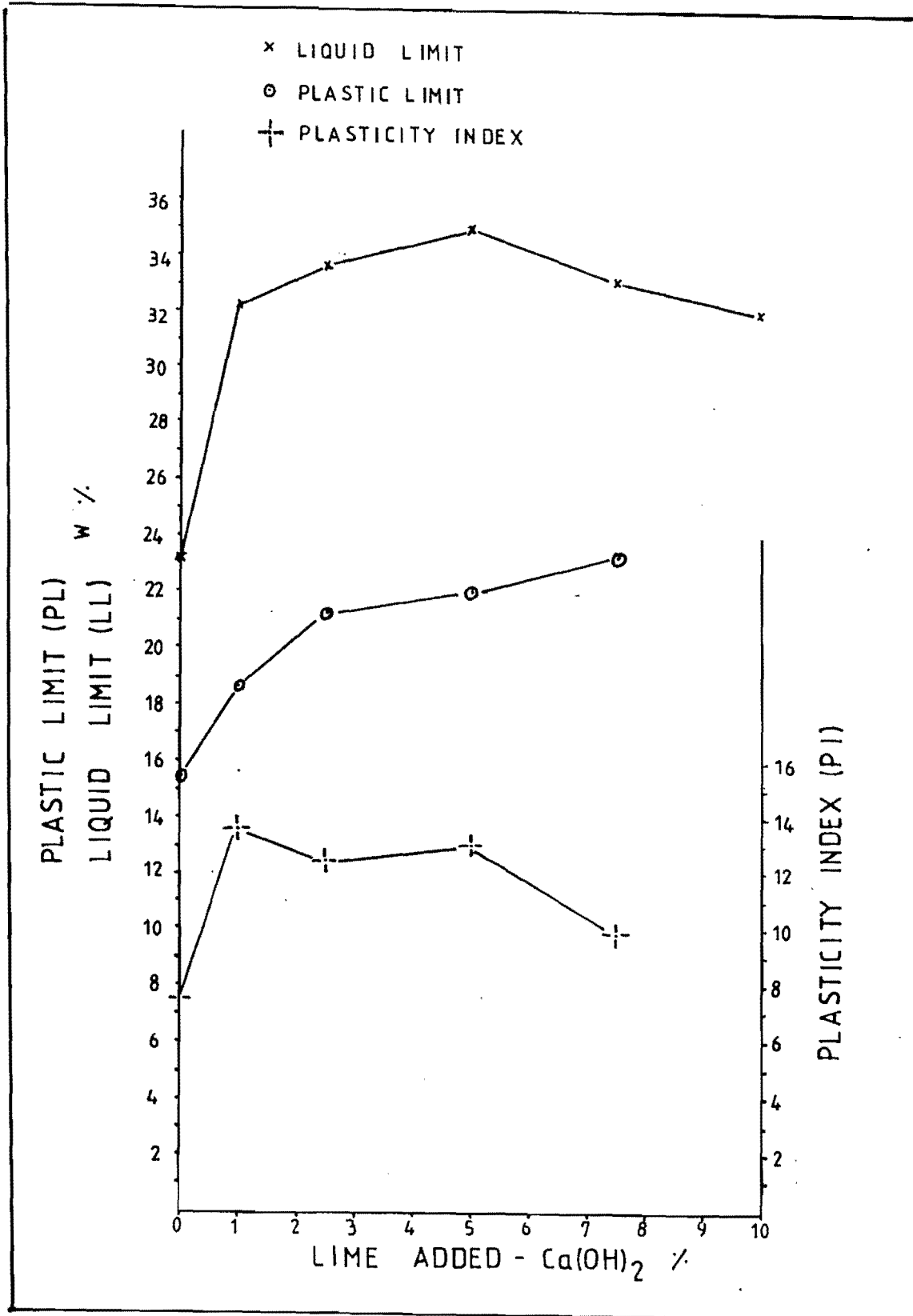


Fig. 3.5. Changes to the liquid limit, plastic limit, and plasticity index, with the addition of lime to the soil. Two samples for each test were averaged.

3.3.6 Optimum moisture content and dry density

Previous work (Herrin and Mitchell, 1961, Neubauer and Thompson, 1972, Alexander et al. 1972) has shown that the optimum moisture content is increased with increasing lime content, and dry density on compaction is reduced with increasing lime content. Evans and Bell (1981) reported that the optimum moisture content, of the loess soil tested, was increased by 3% with the addition of 5% lime. Maximum dry density was reduced by 0.07 t/m^3 at the same lime percentage.

Fig. 3.6. shows the effects of lime on the optimum moisture content and dry density of the soil tested in this study. The compacted dry density of the soil initially decreases sharply with the addition of 1% lime, and is followed by a continued decrease at a slower rate with higher percentages of lime. The dry density of the soil is reduced by 0.18 t/m^3 with the addition of 10% lime. Correspondingly, the optimum moisture content increases in almost the same way as the maximum dry density decreases, with the optimum being increased from 13.3% for the untreated soil to 17% for the soil treated with 10% lime.

The flocculation of clay particles and overall increase in grainsize with the addition of lime is partly responsible for the decrease in dry density. The increase in optimum moisture content is due to the property of lime stabilised soils to release and adsorb less water than natural soils, (Brandl, 1981, Brand and Schonenberg, 1959) which requires that additional water be added to the soil to give maximum densities on compaction.

3.3.7 Erodibility and dispersion

The pinhole erodibility test of (Sherard et. al. (1976), as modified by Evans (1977), (see appendix 3 for details of test method) was used to determine the erodibility of the soil. A modified classification system developed by Yetton (1986) which concentrates on measuring the erodibility of the soil rather than "colloidal dispersion" as defined by Sherard et.al. (1976), is used in this study and is detailed in appendix 3.

The Yetton classification does away with the terminology D - ND

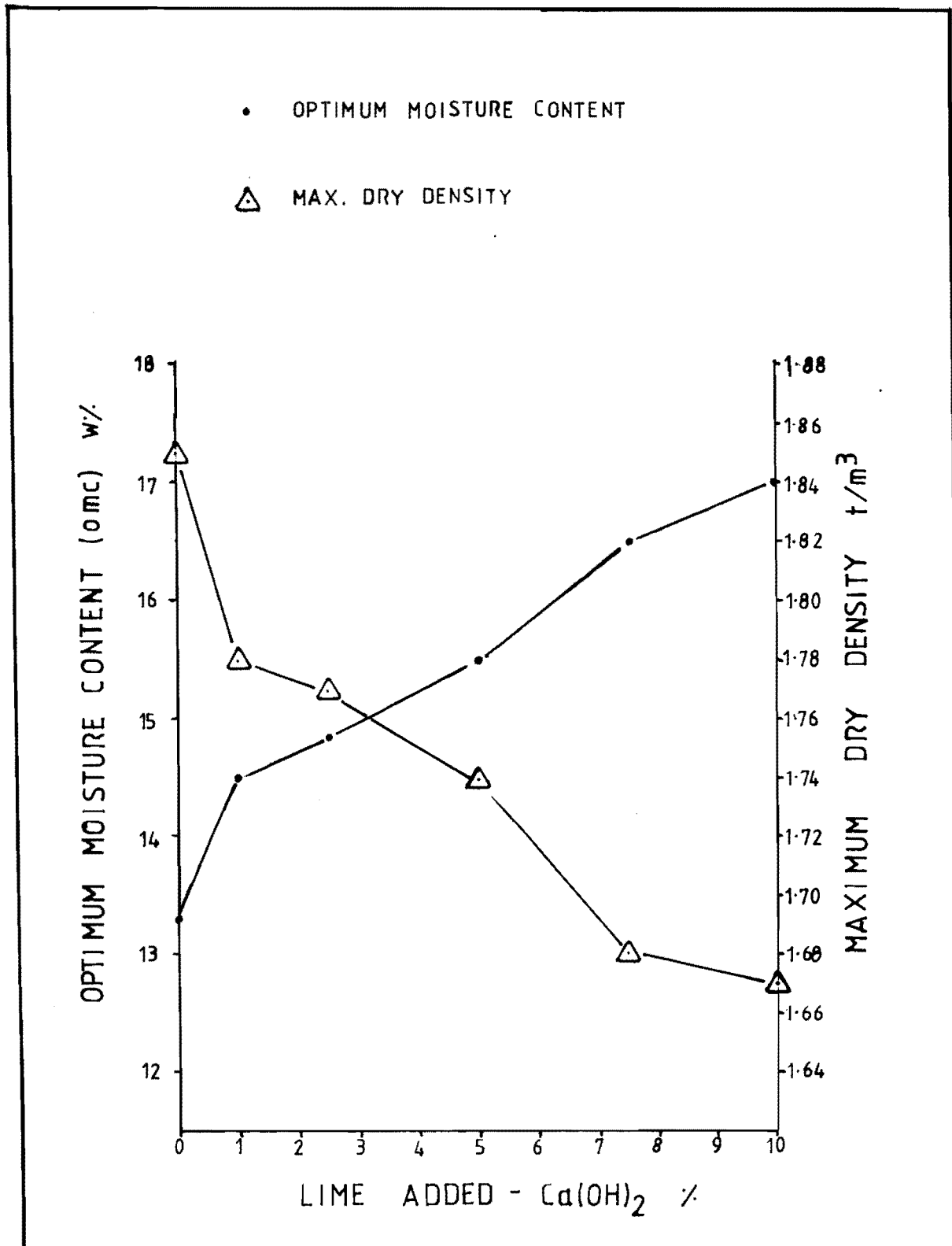


Fig. 3.6. The effect of lime addition on optimum moisture content and dry density.

(dispersive and non - dispersive) and concentrates on determining erodibility, which is not just a factor of dispersion. Yetton introduces the term "sustained erosion" and defines it in terms of increased flow rate over a three minute period. Samples are classified by the head at which "sustained erosion" occurs. For example, an erodible sample that undergoes "sustained erosion" at the 180mm head is designated E (for erodible) and postscripted 180, to give the classification of E 180. Non-erodible samples are designated NE (non-erodible).

The dispersibility of the soil was determined using the Emerson crumb test (Emerson (1967)-see appendix 4).

The field sample from the Westmoreland subdivision (W3) was found to be highly erodibly (E180) and dispersive (class I). The untreated recompacted sample is less erodible than the field sample (E380-1000) and is moderately dispersive (class II). The erodible and dispersive nature of the untreated soil makes it susceptible to the erosion problems described in chapter 2.

Evans and Bell (1981) reported the addition of 1% lime rendered all loess samples tested (using the pinhole erodibility test), non-erodible. Machan et al. (1977) reported that the addition of 1% lime to soils tested was sufficient to stabilise the soil against erosion from a 1 hour, laboratory, rainfall test. They found that erosion resistance is better developed with longer periods of curing. They concluded that the erosion resistance was due to the formation of a calcium silicate hydrate gel around some of the soil grains (a permanent, irreversible reaction), which acted as a mesh, linking grains together.

The addition of lime in small percentages (1%) to the soils tested in this study, produced a non-erodible, non-dispersive material (see table 3.3 and appendices 3 and 4). Dispersion is reduced with the addition of lime due to clay minerals forming into meta - stable flocs produced by the "rapid amelioration" effects described in section 3.2.1. Brand and Schonenberg (1959) found that lime stabilised flocs were water resistant and acquired hydrophobic properties, making them less susceptible to dispersion. Physical

erosion is reduced by the formation of reaction products that cement the soil skeleton (as described above and in section 3.2.1).

The production of a non-erodible material at 1% lime is in agreement with the findings of Evans (1978), and Evans and Bell (1981). The production of a non-erodible and non-dispersive material is important for loessial soils, as it prevents clay mineral dispersion and physical erosion of the soil skeleton by moving water, both of which are major processes involved in the formation of tunnel gullies, and contribute to sheet and rill erosion on the Port Hills.

	Lime content %					
	0	1	2.5	5	7.5	10
Pinhole Erodibility	E 380- ¹ 1000	NE	NE	NE	NE	NE
Emerson dispersion class	II	IV	IV	IV	IV	IV
Swelling	² 8.4	4.4	2.8	1.25	0.5	0.2
Strain	³ 6.8	3.5	2.25	1.03	0.48	0.12
E _s %	⁴ -	4.75	3.25	1.4	0.51	0.03

1 Range of erodibility of 2 samples.

2 Tested at omc, 14 days moist cured.

3 Tested at air dried state.

4 Tested at soaked state

Table 3.3 Erodibility, dispersion, and swelling strain data for the untreated soil and lime treated soil.

3.3.8 Swelling strain

Previous research (Brandl, 1981, Herrin and Mitchell, 1961, Evans and Bell, 1981) has shown that volume changes (on saturation and drying) of lime stabilised soils is reduced. Evans and Bell (1981), (see fig. 3.7) found that swelling strain of loess was minimised at 2.5% lime. The reduction of volume changes is important for Port Hills loess, as shrinkage and swelling contribute to the formation of tunnel gully erosion.

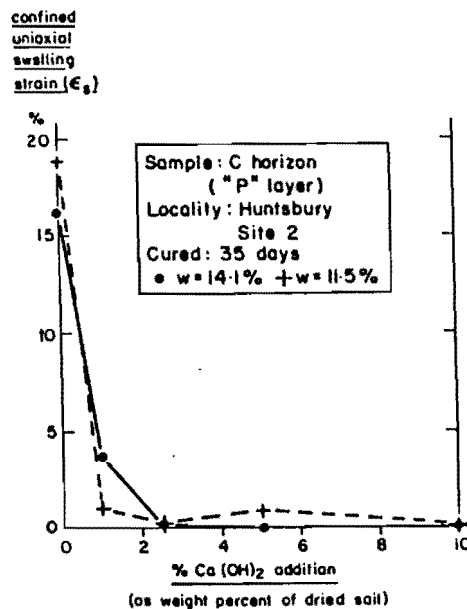


Fig. 3.7. Confined uniaxial swelling strain data for lime stabilised parent-loess samples, Port Hills (Evans and Bell, 1981).

The uniaxial swelling strain was ascertained for the field sample (W3) and for the untreated, recompacted 14 day moist cured and 7 day air-dried samples. The samples were found to have a moderate to low swelling potential, the recompacted samples having a lower swelling potential than the field sample. The small amount of smectite (swelling clays) in the soil, the disruption to the soil skeleton by increased pore water pressures, and the reduction of soil moisture suction on saturation, are responsible for the swelling behaviour of the soil. These effects are adequate to weaken the bonding of the natural soil and enhance soil erosion.

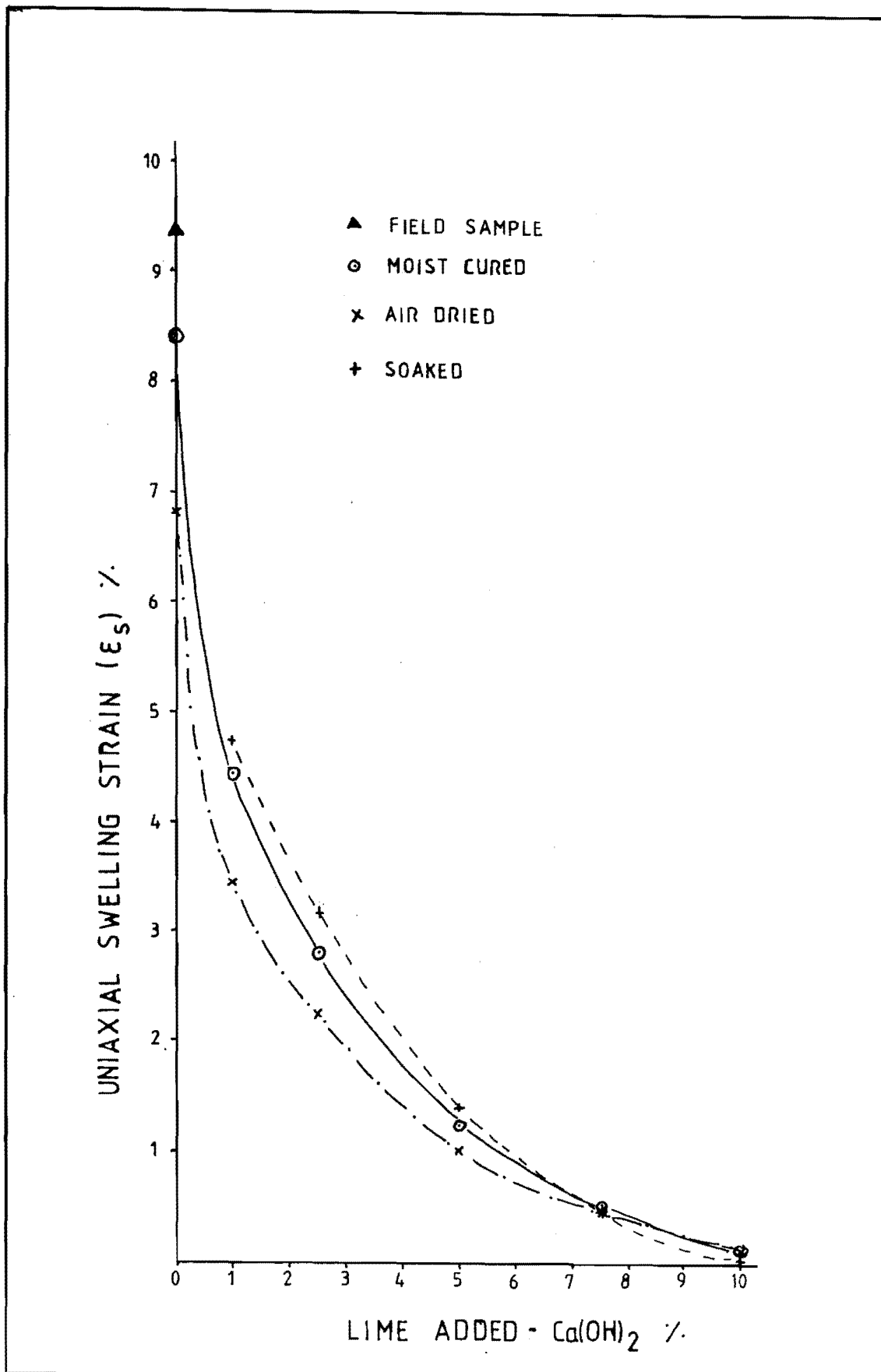


Fig. 3.8. Swelling strain for the field sample from Westmoreland, and the effects of lime addition on swelling strain for the three curing methods.

The swelling potential of the soil is significantly reduced with increasing amounts of lime addition (see fig. 3.8.). Swelling rates are minimised on the addition of 7.5% and 10% lime. The addition of lime to the soil decreases swelling potential due to increased cementing of the soil skeleton and increased osmotic pressures (chemical solutions of higher densities than water in isolated pores) which tend to draw the soil particles together. The increase in grain size with the addition of lime, increases pore space, meaning that lower pore pressures are exerted on the soil structure on saturation, and the effects of soil moisture suction are reduced.

However, the reduction of swelling strain for the samples tested do not compare favourable with the results of Evans and Bell (1981), and Brandl (1981). Both showed that the addition of between 1% and 3% lime minimised swelling. Evans and Bell allowed their samples to air-equilibrate for a period of 28 days before testing. The possibility of a slow or delayed reaction of the soil with low lime percentages (1% - 2.5%, as discussed in sections 3.3.4 and 3.3.5), did not allow samples, cured for 14 days, enough time to properly form reaction products that significantly reduce swelling. It is noticeable, from the results shown in figure 3.8., that up to 5% lime, air dried samples have lower swelling potentials than the other curing methods.

Swelling strains increase at higher moisture contents (at the time of testing - refer to fig. 3.9) for the untreated soil and for the soil treated with up to 5% lime. At higher percentages of lime (7.5% and 10%), the swelling potential remained constant with increasing moisture content (at the time of testing). This is due to the increased osmotic pressure of lime stabilised soils and indicates that lime stabilised soils release and adsorb water less readily than untreated soils.

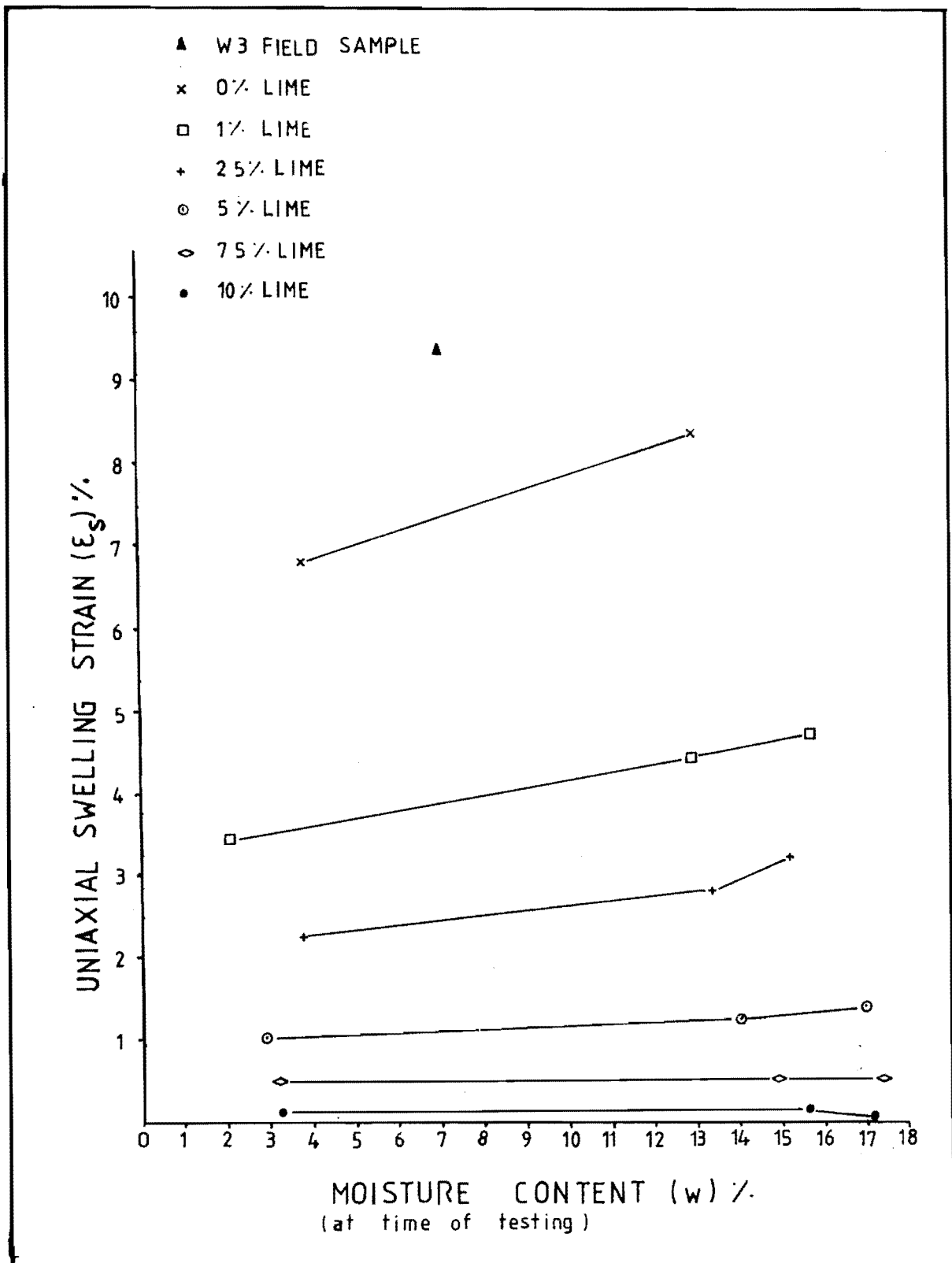


Fig. 3.9. The influence of moisture content at time of testing on swelling strain for the field sample from Westmoreland, and the soil treated with lime.

3.3.9 Slaking

Slaking is an important process in the formation of tunnel gullies and other erosion problems encountered with Port Hills loess. Lime stabilised loess has also been used as linings for stormwater drains and watercourses of intermittent flow (Bell, 1982a and 1982b). For this reason, moulds were subjected to 7, 24 hour wetting and drying (at room temperature) cycles.

Five untreated, recompact samples, moist cured for 7 days, were then subjected to the wetting and drying cycles. The untreated soil was immediately affected by slaking when subjected to the first wetting cycle (see figs. 3.10. and 3.11.) and had completely slaked by the end of the 7 day period.

Slaking of soil is caused by changes to fluid induced stresses; increased pore water pressures and the reduction of soil moisture suction on saturation. These stresses are sufficient to disrupt the soil fabric and weaken soil bonds.

Some slaking occurred to the soil treated with 1% and 2.5% lime (see figs. 3.12. and 3.13.). Machan et al. (1977) found that for some soils, 1% lime was not completely effective on minimising slaking. The soil treated with 5%, 7.5%, and 10% lime survived the slaking period with only minor erosion around the top edge of the mould.

The addition of lime to the soil has similar effects on reducing slaking as it does for reducing swelling potential. That is, increased grain size lowers pore pressure on saturation and osmotic pore pressures are increased. The formation of cementing products (as discussed in section 3.2.1) increases the strength of the soil skeleton, enabling the soil to withstand the effects of positive pore pressures on saturation.



Fig. 3.10 Untreated samples (0% lime) in the slaking tank after three days wetting and drying.



Fig. 3.11 One untreated sample (0% lime) of soil after three days of wetting and drying. The original size of the sample is outlined.



Fig. 3.12 The effects of 7, 24 hour wetting and drying cycles on a sample treated with 1% lime. Note slaking around the edges of the mould and that the three compacted layers stand out.



Fig. 3.13 The effects of 7, 24 hour wetting and drying cycles on a sample treated with 2.5% lime. Minor slaking has occurred around the perimeter of the mould. Moulds treated with 5%, 7.5% and 10% lime suffered only minor slaking around the top edge of the mould.

3.4. SYNTHESIS

3.4.1 General

(i) The addition of lime to the soil in small percentages (1%) produces an erosion resistant, non-dispersive material.

(ii) Slaking and swelling strain are reduced by the initial addition of lime and minimised by the addition of 5% and 7.5% lime respectively.

(iii) Optimum moisture content is increased and maximum dry density is decreased with increasing amounts of lime.

(iv) Grainsize analyses show that the addition of lime increases the grainsize of the soil, due to the flocculation of clay minerals.

(v) Although the soil is more "workable" with the addition of lime, due to an increase in grainsize (pulverisation and mixing become easier), the plasticity of the soil is increased. There appears to be a certain amount of lime required (5 - 7.5%) to start to significantly reduce the plasticity of the soil. This is probably a result of the slow response of lime to the relatively unreactive mixed layered illite clays present in the soil which is also indicated by the slow reduction of swelling potential.

3.4.2 Further Investigations

There seems to be an unusual or delayed initial reaction of the soil (indicated by unusual changes to grainsize, the increase in plasticity of the soil and slow reduction of swelling strain (cf. Evans and Bell, 1981)), with the addition of between 1% and 2.5% lime. The response of lime added in small percentages (1% -2.5%) to the soil requires further investigation. Further investigation should utilise X-ray analysis and Scanning Electron Microscope techniques to determine the type of reaction and reaction products at these low lime percentages.

CHAPTER FOUR

STRENGTH CHARACTERISTICS OF LIME STABILISED LOESS

4.1. INTRODUCTION

The addition of lime to a soil in small percentages (between 1%-12% of dried weight of soil) has the effect of increasing its strength. The amount of strength gain is variable and depends on properties of the, soil and the conditions of stabilisation. Herrin and Mitchell (1961) recognise six major factors that effect the strength of lime - soil mixtures;

- 1) lime content,
- 2) type of lime,
- 3) type of soil,
- 4) density (or compaction),
- 5) type of curing, and
- 6) time of curing.

These are discussed further in section 4.2.

Strength gains of between 2 and 15 times the strength of the original soil have been reported with the addition of lime (Neubauer and Thompson, 1972, Remus and Davidson, 1965, Brandl, 1981, Evans and Bell, 1981). Evans and Bell (1981) found that the addition of 5% lime to loessial soils from the Port Hills optimised strength gains (500 - 900 kPa), six times the strength of the untreated soil.

Strength characteristics of a loessial soil treated with hydrated lime and cured under various conditions are discussed in this chapter. Uniaxial unconfined compressive strength and axial strain deformation during compression were measured. The soil tested is from the Westmoreland subdivision and details of property changes (excluding strength) of the soil with the addition of lime are given in chapter 3.3.

4.2. STRENGTH GAINS OF LIME - SOIL MIXTURES

4.2.1 Lime content

It is well established phenomena that strength increases occur to a soil with increasing lime content up to a certain percentage (there are some exceptions, eg. soils with a high organic content). Many researches have found that the addition of lime beyond this optimum amount gives a reduction in strength. The amount of lime that has to be added to give optimum strength gains varies from soil to soil and with the type of lime that is used (refer table 4.1. and figs. 4.1 - 4.3).

Neubauer and Thompson (1972) found that the addition of 6% lime gave optimum strength gains, while Brandl (1981) reported that 7.5% lime gave optimum gains in strength. Evans and Bell (1981) report that the addition of 5% lime to loessial soils of the Port Hills optimised strength (fig 4.2). Table 4.1. (from Clare and Cruchley (1957) after Levchanovskii (1952)) shows the optimum quantities of lime required to stabilise soils with different textural types.

Table 4.1. Optimum lime percentages to stabilise soils of different textural types.

	Optimum lime content %
Sandy clay soils, and mechanically stable soils	4 - 5
Light- and medium textured silty and clayey soils	6 - 7
Heavy silty and clayey soils	7 - 8

Alexander et al. (1972) report strength losses as the lime content is increased beyond the optimum. They conclude that the losses are due to reduction of dry density on compaction. However, Brandl (1981) suggests that strength losses beyond the optimum are due to additional lime not reacting with the soil, and that the soil particles "swim" in the gel like substance of the unreacted or partially reacted lime.

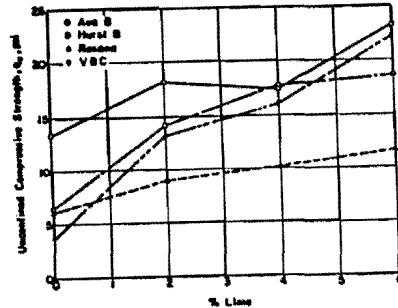


Fig. 4.1. Immediate effects of lime treatment on unconfined compressive strength (Neubauer and Thompson, 1972).

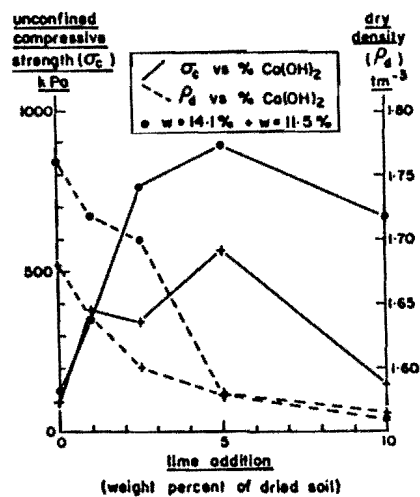


Fig. 4.2. Unconfined compressive strength and compacted dry density plots for lime additions to parent-loess, Port Hills (Evans and Bell, 1981).

4.2.2 Type of lime

Research by Lu et al. (1957), Remus and Davidson (1961), and Wang et al. (1963) indicates that various types of lime have different effects on strength gains for different soils. These reserchers found that dolomitic limes give higher strengths than calcitic limes and that dolmitic quicklimes give the highest strength gains for most soils. They attributed this to the greater hydration potential of quicklimes. Figure 4.3. shows the effects of various forms of commercial lime on strength gains of a friable and plastic loess (Lu et al., 1957).

Alexander et al. (1972) also found that quicklime gave higher strength gains than hydrated limes. They also reported that granular limes give higher strengths than fine limes, due to a function of higher attainable densities on compaction.

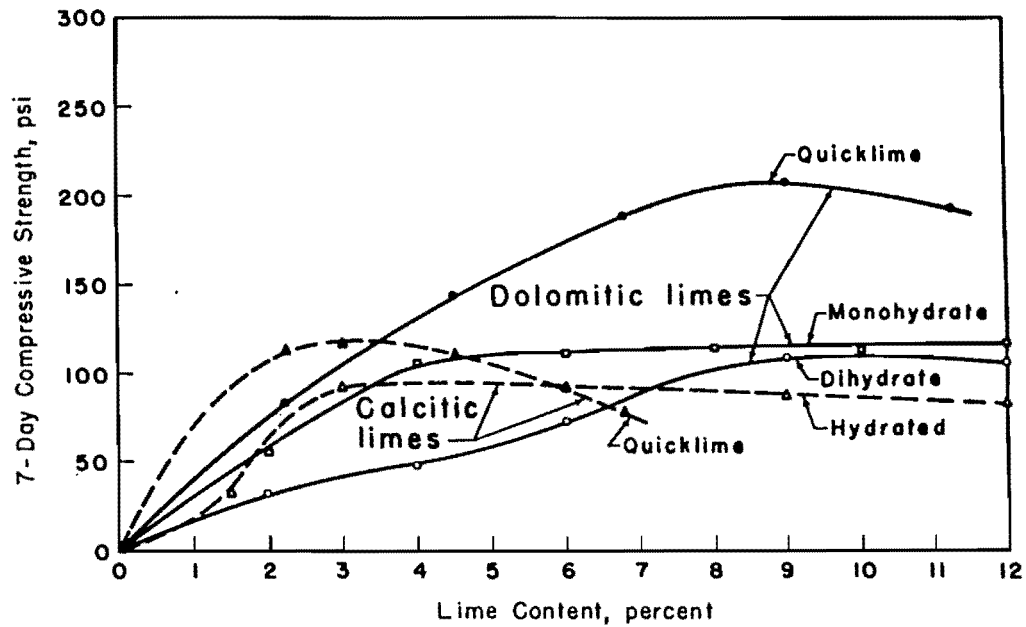
4.2.3 Type of soil

Two requirements for soil stabilisation by lime are that it contains pozzolanic material and clay - sized material ($2\mu\text{m}$). However, only a small percentage (approximately 10%) of clay sized material is required (Herrin and Mitchell, 1961).

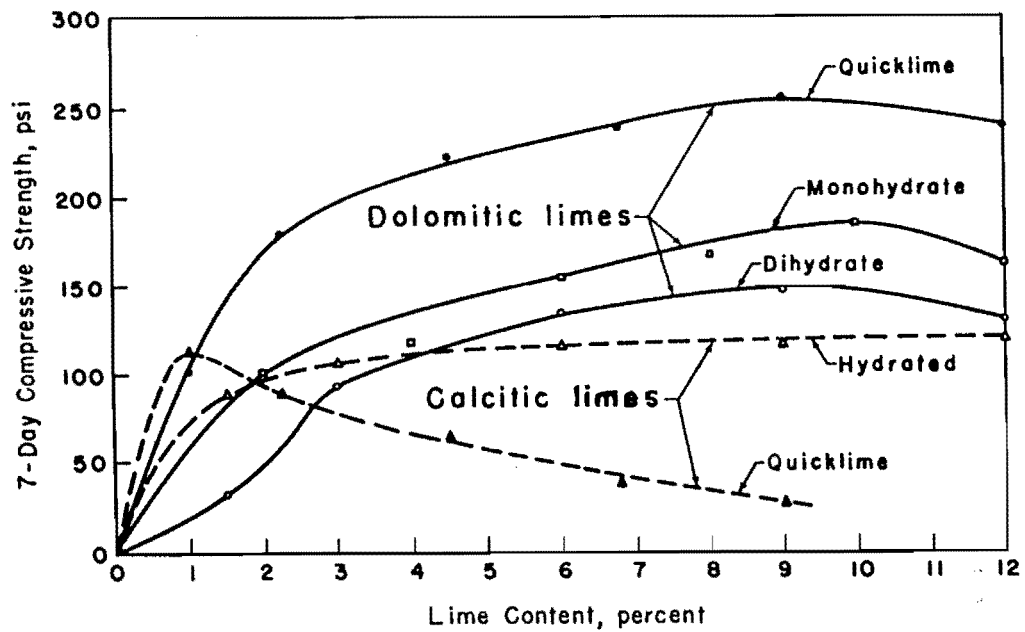
Pozzolans are siliceous materials that react with lime to form cementitious compounds. If natural pozzolanic material is absent from the soil it must be added before stabilisation can occur. Artificial pozzolans that can be added to the soil include, pulverised blast furnace slag, fly-ash (pulverised fuel ash), ground brick, and expanded shale dust (Ingles and Metcalf, 1973).

Figures 4.1. - 4.4 all show the effects of lime on the strength of various soils stabilised with hydrated lime. Table 4.1 shows the optimum amount of lime required to stabilise soils of different textural types.

Mitchell and Herrin (1961), state that "generally the highly plastic soils are more reactive with lime." Eades and Grim (1960) concluded that the strength of kaolin soils increased significantly with low percentages of lime (1% - 2%), whereas for other clay minerals 4% lime addititon is required for significant strength gains.



(a)



(b)

Fig. 4.3. The effect of various commercial limes on the immersed compressive strength of (a) plastic loess and (b) friable loess (Lu et al., 1959). Note that the strength of the soils decrease or are unaffected by the addition of lime beyond the optimum.

Conversely, Croft (1965) found that mixed layered clays showed more rapid early strength gains than the kaolinites, and that montmorillonite clay soils required 5% lime addition to significantly effect strength increases. Croft also found that a high percentage of lime (8% +) in an illite clay soil resulted in strength losses.

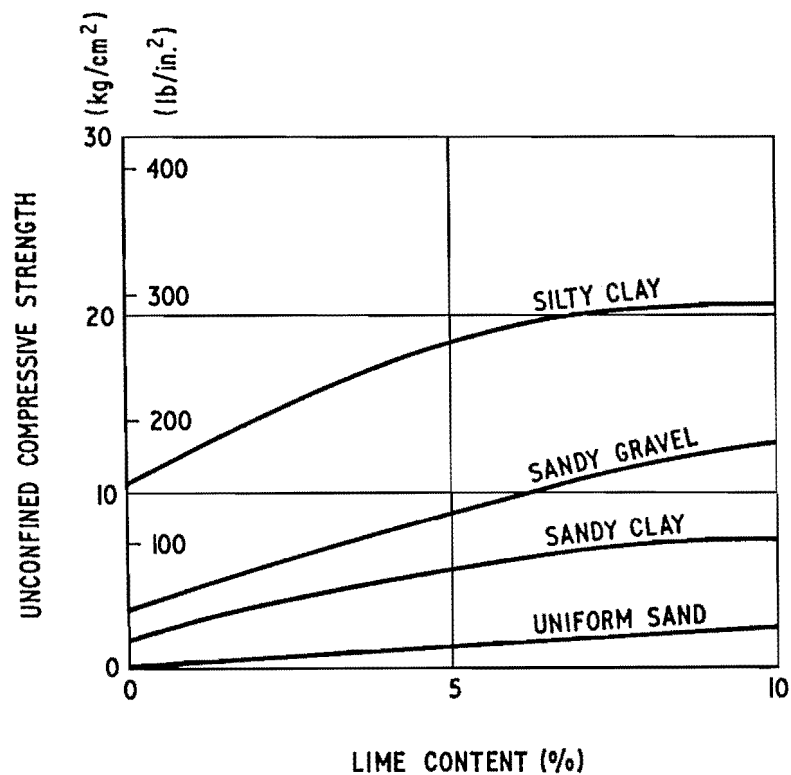


Fig. 4.4. Effect of lime content on strength for various soils stabilized with hydrated lime, cured for 7 days at 25°C, constant moisture content (Ingles and Metcalf, 1973).

4.2.4 Density

The strength of a lime-soil mixture is physically increased when the mixture is compacted, due to an increase in density. Remus and Davidson (1961) found that the AASHO modified compactive effort gave higher strengths than standard compactive efforts. Alexander et. al. (1972) found that on compaction, soils stabilised with quicklimes gave greater densities than other limes which correspondingly led to higher strength gains of quicklime stabilised soils. (see 4.2.2. Type of lime).

4.2.5 Time of curing

Research (Laguros et al. 1956, Herrin and Mitchell, 1961, Remus and Davidson, 1961, Croft, 1964 and Ingles and Metcalf, 1973) shows that the strength of lime soil mixtures increases with age (see fig. 4.5). Brand and Schonenberg (1959) and Laguros et al. (1956) showed that the strength of lime stabilised soils increases rapidly with initial curing time, but there is a decrease in the rate of strength gain as the curing period is extended (see fig.4.6).

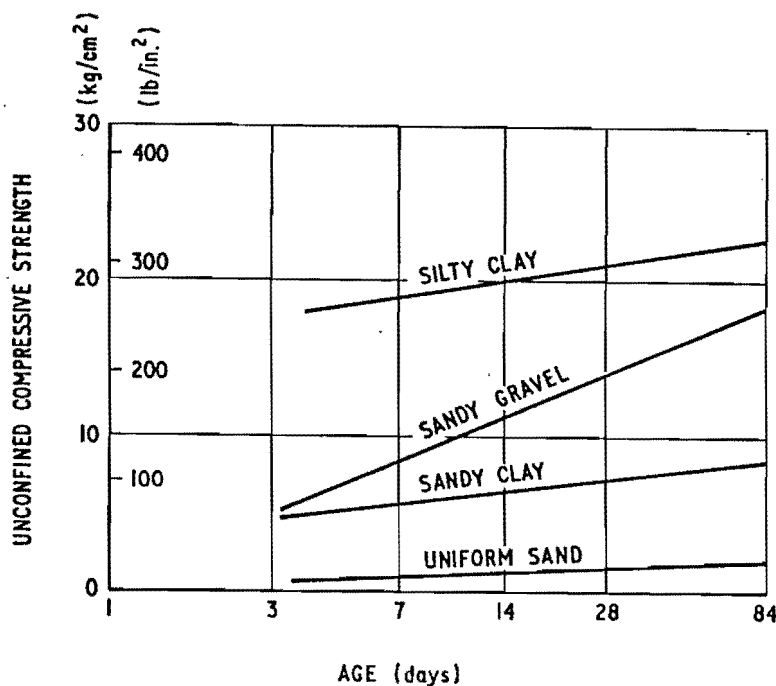


Fig 4.5. Effect of curing time on strength of soils stabilised with hydrated lime, cured at 25°C, constant moisture content (Ingles and Metcalf, 1973).

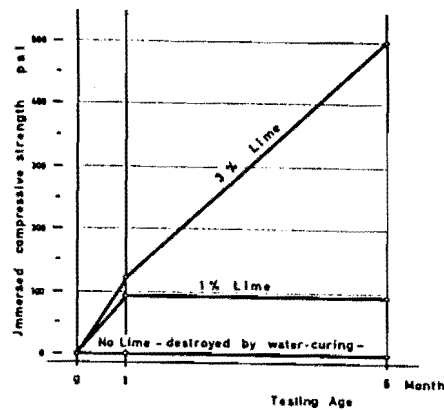


Fig. 4.6. Immersed unconfined compressive strength related to age and amount of lime (Brand and Schonenberg, 1959).

4.2.6 Type of curing

Various methods have been used to cure lime soil mixtures. Primarily the different methods of curing can be divided into two basic groups;

- a) curing at normal or elevated temperatures,
- b) curing at varying moisture conditions and / or relative humidities.

a) Curing at normal or elevated temperatures .

Figure 4.7 (from Ingles and Metcalf, 1973) shows the effect of curing at elevated temperatures. Curing at elevated temperatures has the effect of increasing the rate of strength gain. Anday (1963) found that curing lime soil mixtures for 3 days at 60° C. gave strengths equivalent to 40-45 days of field curing at 16° C. He also found that there is little reaction of lime below 10° C.

Laguros et al. (1956) found that 7 days of curing lime stabilised soils at 110F (48° C) or 140F (60° C) gave a three to eightfold increase in compressive strength over specimens cured at room temperature. They reported that the strength of samples cured for 80 days at 70F (20C) was matched by 7 days of curing at 110F.

Both Anday and Laguros et al. found that the strength of soils cured at 140°F could never be attained by curing at 70°F (see fig. 4.8).

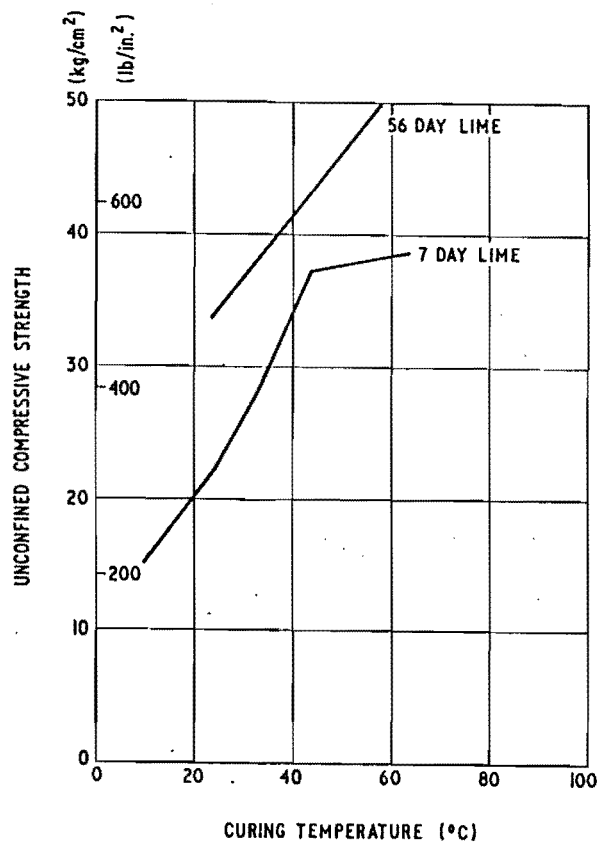


Fig. 4.7. Effect of curing temperature on strength of lime stabilised heavy clay (Ingles and Metcalf, 1973).

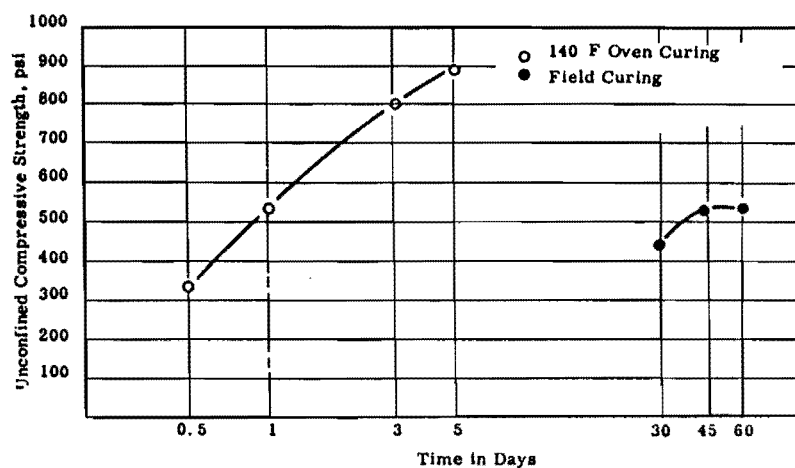


Fig. 4.8. Comparison of field curing and 140°F (60°C) oven curing on a soil stabilised with 5% lime (Anday, 1963). Anday concluded that unconfined compressive strengths of soil cured under field conditions could never match strengths of soils cured at 140°F.

b) Curing at varying moisture content or relative humidity .

The humidity of the air during the curing period appears to have some effect on the strength of lime soil mixtures. The exact effect, and best conditions for curing are unclear (Herrin and Mitchell (1961)). Laguros et al. (1956) found that the curing of moulds at relative humidities of 90% had higher strengths than those cured at lower humidities.

Data indicate that strengths higher than those produced by either moist curing or high temperature curing can be obtained by various combinations of curing methods. Curing at constant relative humidities and normal temperatures (approximately 20°C) is closer to those conditions found in roading pavements or most field situations, and strengths obtained in the laboratory under these conditions give a better indication of likely strength gains that will be achieved in the field. Strength gains, and the rate of strength gains of field cured mixtures, commonly do not match those of mixtures cured under controlled laboratory conditions (see fig. 4.8).

4.3. TEST METHODS

Moulds (105mm diameter x 115mm length) were prepared for the untreated soil and lime treated soil (1, 2.5, 5, 7.5, and 10 percent added by dry weight of soil). The moulds were compacted at optimum moisture content for each lime percent using standard Proctor compaction methods (appendix 8). The unconfined compressive strength of the lime - soil mixtures was the strength parameter measured because of the simplicity of the test, the large number of samples to be tested, and it is one of the more common strength tests used to measure strength gains of lime stabilised soils.

Strengths were tested after curing periods of;

- 1) 14 days moist curing at 20° C and 99% relative humidity,
- 2) 7 days moist curing (as for 1) followed by 7 days air drying,
- 3) 7 days moist curing (as for 1) followed by 7, 24 hour cycles of wetting and drying.

Details of the curing procedure are given in appendix 2. Five moulds

of untreated soil were tested immediately after compaction to act as curing controls.

For each lime percentage, and for each curing period, five samples were tested and the mean taken. Stress/strain curves and details of test procedures are given in appendix 2.

4.4. STRENGTH OF THE UNTREATED SOIL

Stress / strain curves given in figure 4.9. show the averaged unconfined compressive strength (and envelopes of scatter about the mean) of the untreated, uncured soil (control), the untreated moist cured soil (0% lime), and the untreated, air dried soil (0% lime). No soaked strength data could be obtained, as the untreated soil did not survive the cyclic wetting and drying process.

The strength of the untreated soil is almost doubled after a 14 day period of moist curing. However, extraordinarily high strength values are obtained for the untreated soil in the air dried state (μ 4000 kPa), almost 27 times the strength of the control, and 20 times the strength of the untreated soil, moist cured for 14 days (both compacted and tested at omc). High dry strengths of loess are not uncommon on the Port Hills. Many contractors have verbally expressed the problems encountered when excavating in loessial soils during the dry summer months.

The reasons for such high dry strengths can be partially (if not wholly) attributed to increased soil suction of the untreated soil. The effects of increased soil suction (pore water tension or capillary tension) increasing the strengths of soils is well known (Holtz and Kovacs, 1981, Lee, White and Ingles, 1983, Winterkorn and Fang, 1975, Mitchell, 1976). Figure 4.10. shows how soil suction increases with decreasing moisture content and void ratio. Winterkorn and Fang (1975) have shown (fig. 4.11) that decreasing moisture content increases penetration resistance (strength) of cohesive soils due to increased soil suction.

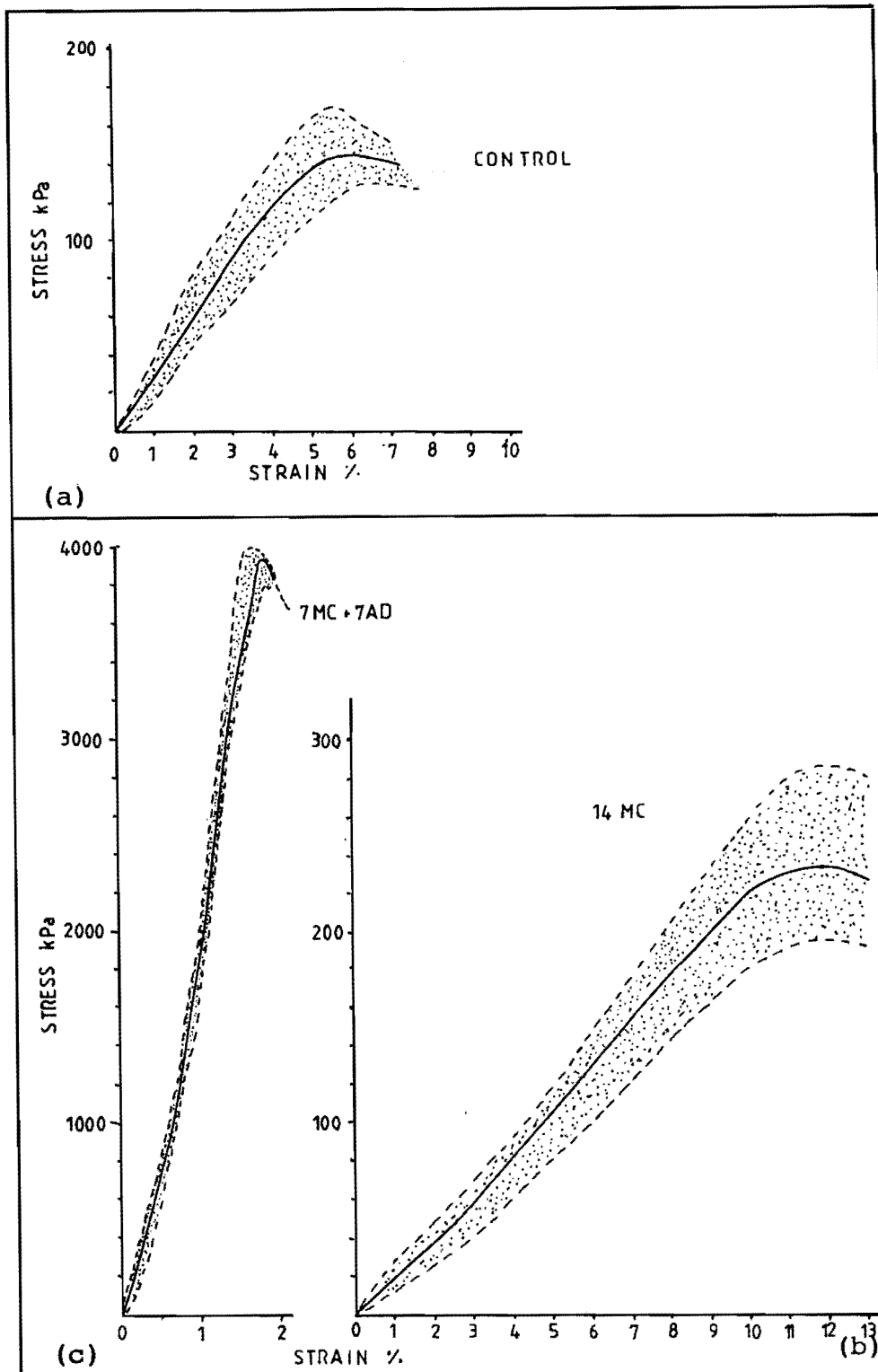


Fig. 4.9. Averaged stress/strain curves (and envelopes of scatter about the mean) for (a) the control (untreated, uncured samples), (b) the untreated soil, moist cured for 14 days and (c) the untreated, 7 day moist cured + 7 day air dried sample. Note that the strength of the moist air dried sample is ≈ 20 times the strength of the moist cured sample and almost 27 times the strength of the control sample.

Boosinsuk and Yong (1982) report that unsaturated residual soils in Honk Kong have sufficient shear strength, attributed to soil suction, to enable them to stand on steep slopes. As the soils become saturated (due to tropical rainstorms) strength is progressively reduced which results in lack of shear strength and general slope failure. Mitchell (1976) reports that loess deposits of the USA have high strengths and are reasonably incompressible in a dry state. However, when saturated the loess deposits lose their strength and stability.

The untreated soil did not survive the wetting and drying process (and therefore has nil compressive strength) indicating that soil suction is negated on saturation and the effects of increased pore water pressure are sufficient to disrupt the soil skeleton and cause a complete loss of strength.

Increased soil suction can also produce a secondary type of compaction (Brandl, 1981). As the soil dries out, the particles are brought closer together by the effects of increased soil suction and a densification of the soil takes place. The untreated air dried samples show an appreciable (7%) decrease in volume which would result in a substantially higher dry density than that at the time of compaction.

The discovery of such high dry strengths of natural loess is an aside from the main aims of this thesis and requires extensive further investigation.

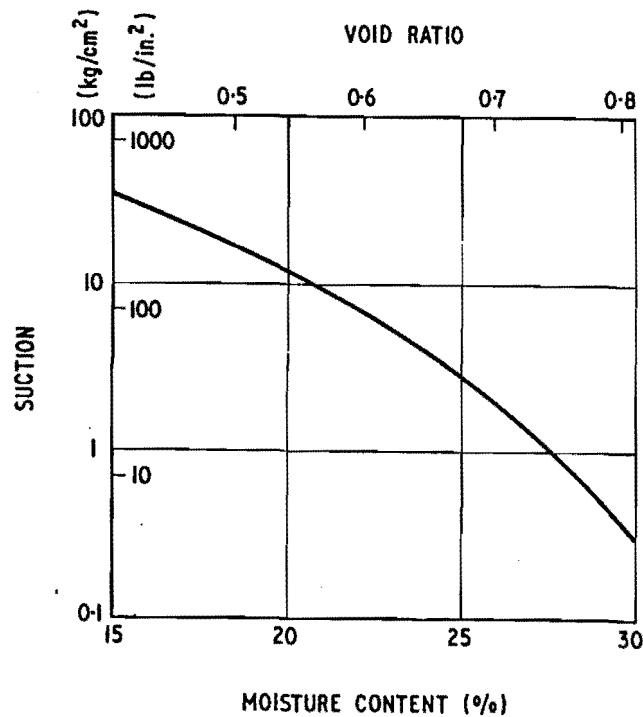


Fig. 4.10. Typical moisture content - matrix suction relationship for a heavy clay (Ingles and Metcalf, 1973).

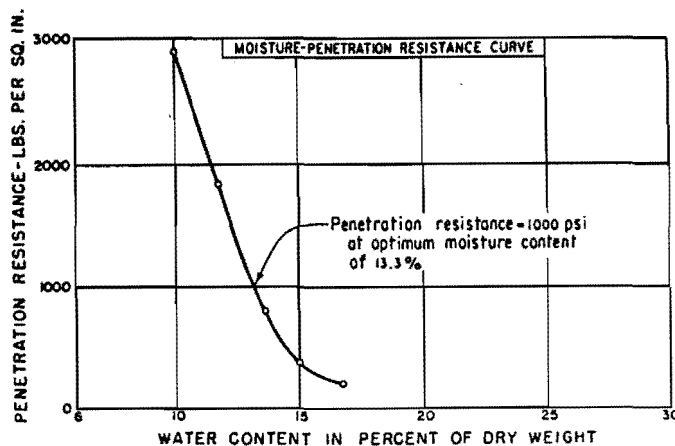


Fig. 4.11. The effects of decreasing moisture content on penetration resistance of a cohesive soil (from Winterkorn and Fang, 1975). If the curve were extrapolated to a moisture content of $\approx 3\%$ then penetration resistance would be increased by a factor of 7.

4.5. STRENGTH OF LOESS TREATED WITH LIME

4.5.1 Strength of moist cured samples

From the data given in figure 4.12., small percentages of lime (1%) have the initial effect of increasing the strength of the soil. This initial increase is followed by a decrease in strength with the addition of 2.5% and 5% lime.

Strength gains for this curing method appear to be optimised at 7.5% lime, with strength losses occurring with the addition of 10% lime. The maximum unconfined compressive strength of 870 kPa at 7.5% lime, is more than three times the strength of the untreated soil (0% lime) for the same curing period and six times the strength of the control sample.

Evans and Bell (1981) found for one of the loess soils they tested, that the addition of 1% lime tripled the strength of the soil. With the addition of 2.5% lime, a decrease in strength was recorded (see fig. 4.2), similar to the response of the soil tested in this study. However, Evans and Bell found that strength of loess soils tested was optimised with the addition of 5% lime whereas for this study strength of the lime treated samples is minimised at 5%. Evans and Bell also found the strength of both soils tested is reduced with the addition of 10% lime.

From the strength data of Evans and Bell (1981) and data obtained in this investigation, it is evident that the addition of 1% lime has a reaction with some loess soils that initially increases its strength, followed by a delayed, or undesirable reaction with the addition of between 2% and 5% lime, that results in strength losses.

Eades and Grim (1960) found that the compressive strength of illite and low swelling Ca^{2+} montmorillonite clay soils decreased with the addition of small percentages of lime (up to 4%). With the aid of X-ray analysis, they found, that although the products responsible for strength gains were present in these soils, they were not well formed at low lime percentages. They concluded that mixed layered clay soils (illites) and varieties of montmorillonite clay soils were slow to react with lime, and that significant strength gains were only

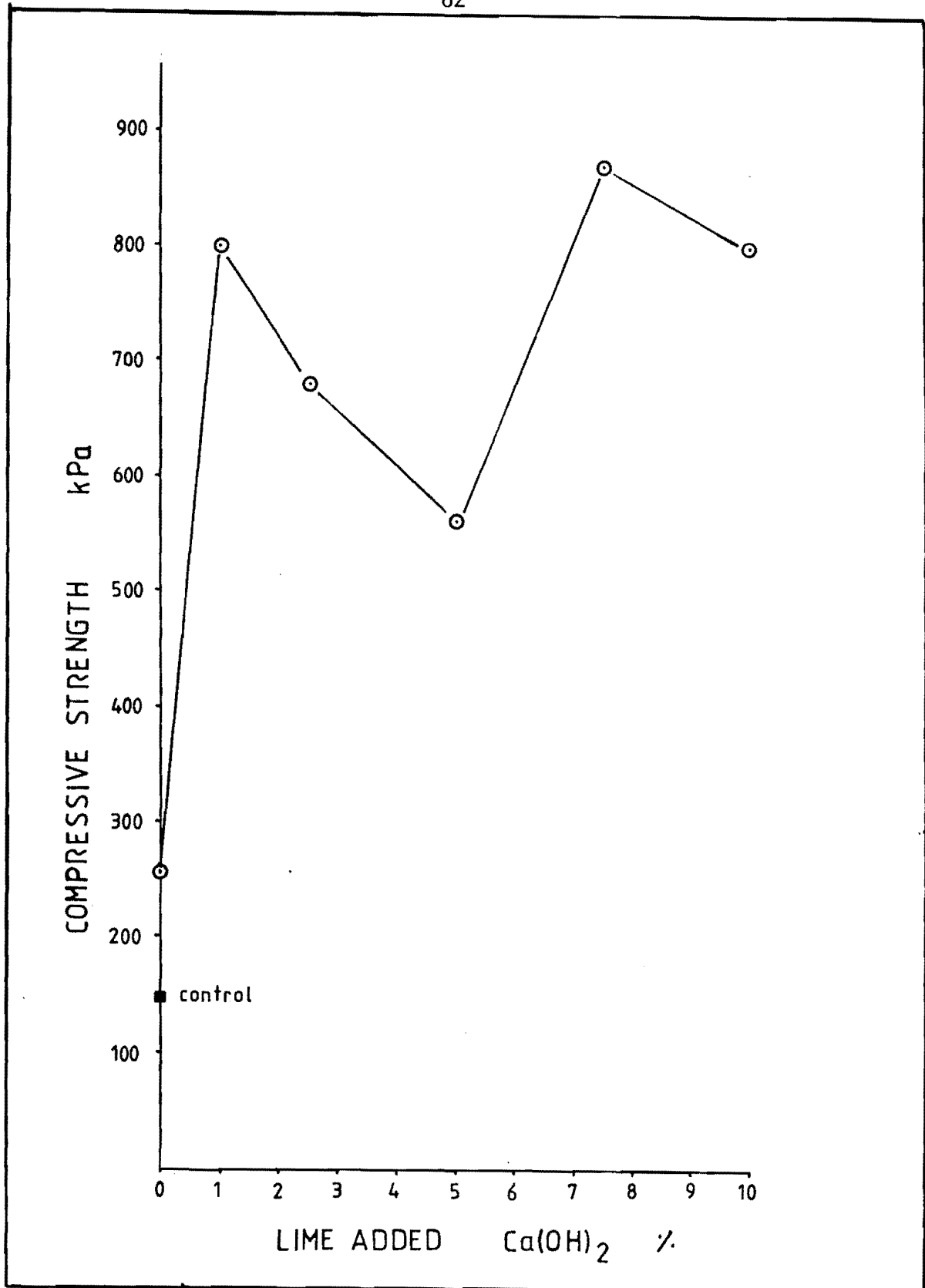


Fig. 4.12. Averaged maximum compressive strengths of the control sample, and lime treated samples, 14 days moist cured.

obtained with the addition of between 4% and 6% lime. Although this may account for the strength losses of 2.5% and 5% lime, it does not explain the high strength gains obtained with the addition of 1% lime.

4.5.2 Strength of air dried samples

Figure 4.13 shows the maximum unconfined compressive strength for the soil moist cured for seven days followed by 7 days air drying. The high strength value obtained for the air dried untreated soil is almost three times greater than the strength of the soil treated with up to 10% lime and many times greater than the uncured control samples.

The high strength of the untreated soil is attributed to high soil moisture suction. The effect of lime on the soil is to increase the grain size of the soil, presumably increasing the void ratio and thereby reducing the soil suction (see fig. 4.10). Clare and Cruchley (1957) found that samples treated with 4% lime held more moisture at a given suction than did untreated soils. This indicates that lime treated soils hold more water in their pores, reducing soil suction and increasing pore water pressures. However, the fact that the strength of the untreated soil is higher than the soil treated with up to 10% lime, indicates that the effects of soil suction must be greater than the bonding or cementation of the reaction products formed on the addition of lime. Although the untreated soil shows exceptionally high dry strengths, it must be remembered that the soil has no strength in the saturated state.

As with the moist cured samples, the air dried samples display a loss of strength with the addition of 2.5% and 5% lime followed by an increase in strength with higher percentages of added lime (7.5% and 10%). The maximum strength for this curing method is obtained with the addition of 1% lime. However the optimum lime percentage to give maximum compressive strength is probably in excess of 10% and testing did not continue above this value.

The maximum strength of the air dried, lime treated soil represents a strength gain of 2 times the maximum strength of the lime treated, moist cured samples, and 4 times the maximum strength of the

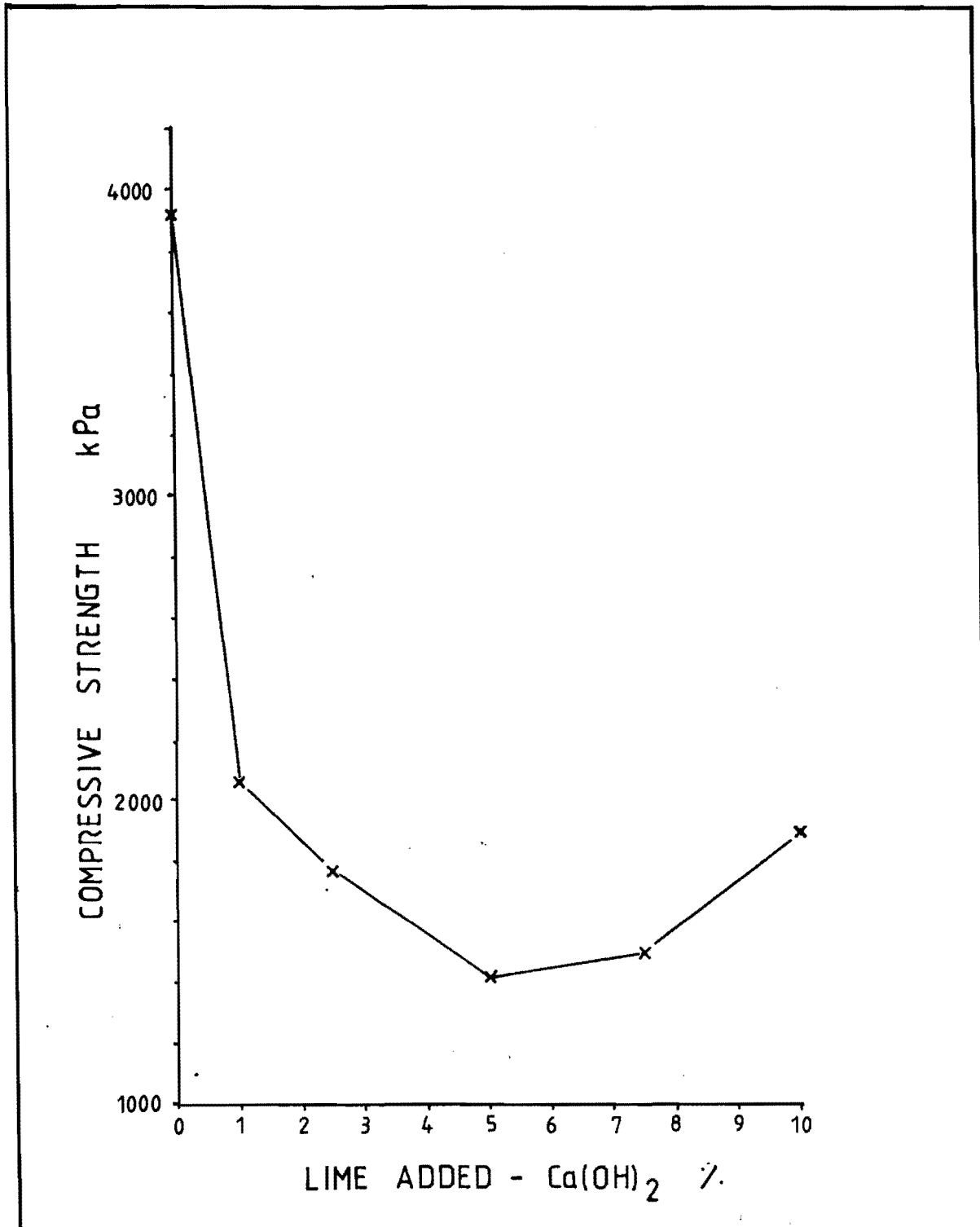


Fig. 4.13. Average maximum compressive strengths for the untreated soil and lime treated soils, 7 days moist cured + 7 days air dried.

lime treated, soaked samples. Ignoring the effects of soil suction and pore water pressures, it is possible that carbonation of the lime, due to exposure to the atmosphere, is a contributing factor in improving the strength of lime stabilised soils. However the soaked samples were exposed to the atmosphere during the drying phases of the wet / dry curing period and the strength gains are lower than those of the moist cured samples, where exposure to the atmosphere was kept to a minimum. It is possible that carbonation is hindered under immersed conditions (Brandl 1981) or has little effect on strength gains. These results prove inconclusive as to the effects of carbonation on improving strength gains, but provide the basis for further investigation in this area.

4.5.3 Strength of soaked samples

Figure 4.14. gives the maximum unconfined compressive strength of lime treated soil tested after an initial 7 day moist curing followed by 7, 24 hour wetting and drying cycles. The moulds were tested in a soaked state.

The untreated soil did not survive the slaking process, the saturation of the soil substantially reducing the effects of soil suction, and the increased pore water pressure disrupting the soil skeleton leading to a complete loss of strength. However the addition of small amounts of lime (1%) enabled the soil to withstand the wetting and drying cycles.

Results show that there is an initial increase of strength with the addition of up to 2.5% lime. As with the other curing methods, this is followed by a decrease in strength with the addition of 5% lime. Lime addition above 5% increases the strength of the soil with the highest unconfined compressive strength (480 kPa) being obtained on the addition of 10% lime. This represents a strength gain of 4 times the strength of the control (untreated, uncured) sample. As with the air dried samples, it is possible that the optimum lime percentage to give maximum strength gains is greater than 10%.

Cyclic wetting and drying of the soil reduces strength gains. The maximum strength obtained for this curing period is almost half that of the moist cured strength and 4 times lower than the maximum

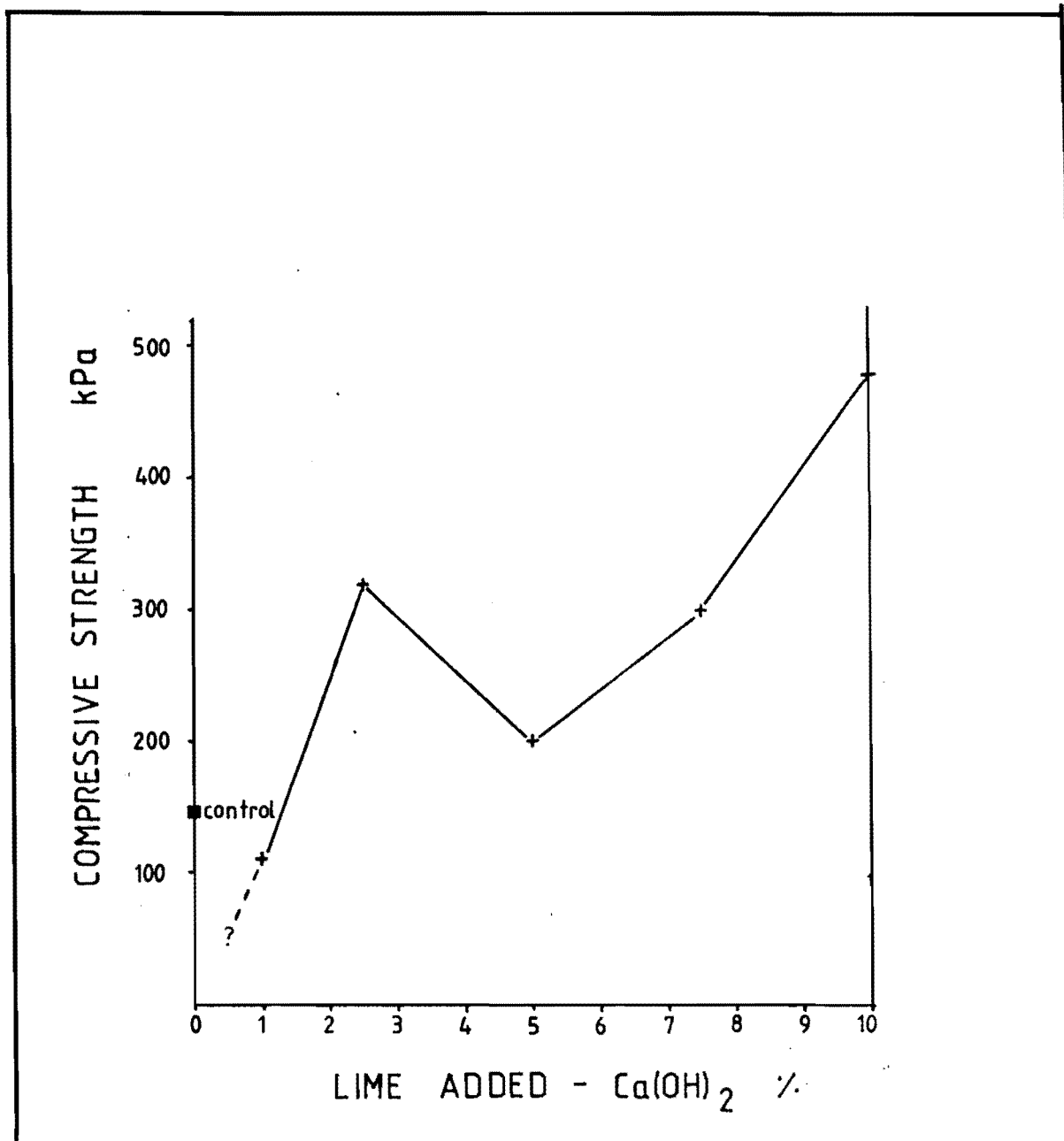


Fig. 4.14. Average maximum compressive strengths for the control sample and lime treated soils, 7 days moist cured + 7 days wet/dry cycles. Note that samples were tested in the soaked state.

strength of the air dried samples. It appears that the immersion of lime treated soils hinders or slows the formation of the compounds responsible for strength gains. Brandl (1981) reports that immersed strengths of lime stabilised soils are reduced, partly due to the hinderance of carbonation in the presence of water.

Figure 4.15 shows the averaged moisture contents of the samples at the time of testing. The moisture contents of the soaked samples are only up to 5% higher than the moisture contents of the moist cured samples. This extra moisture must be sufficient to raise the pore water pressure of the soil and consequently lower the strength of the soil. The effects of osmotic pore pressures (tension effects of chemical solutions in the pores formed on the addition of lime) appear to be minimal.

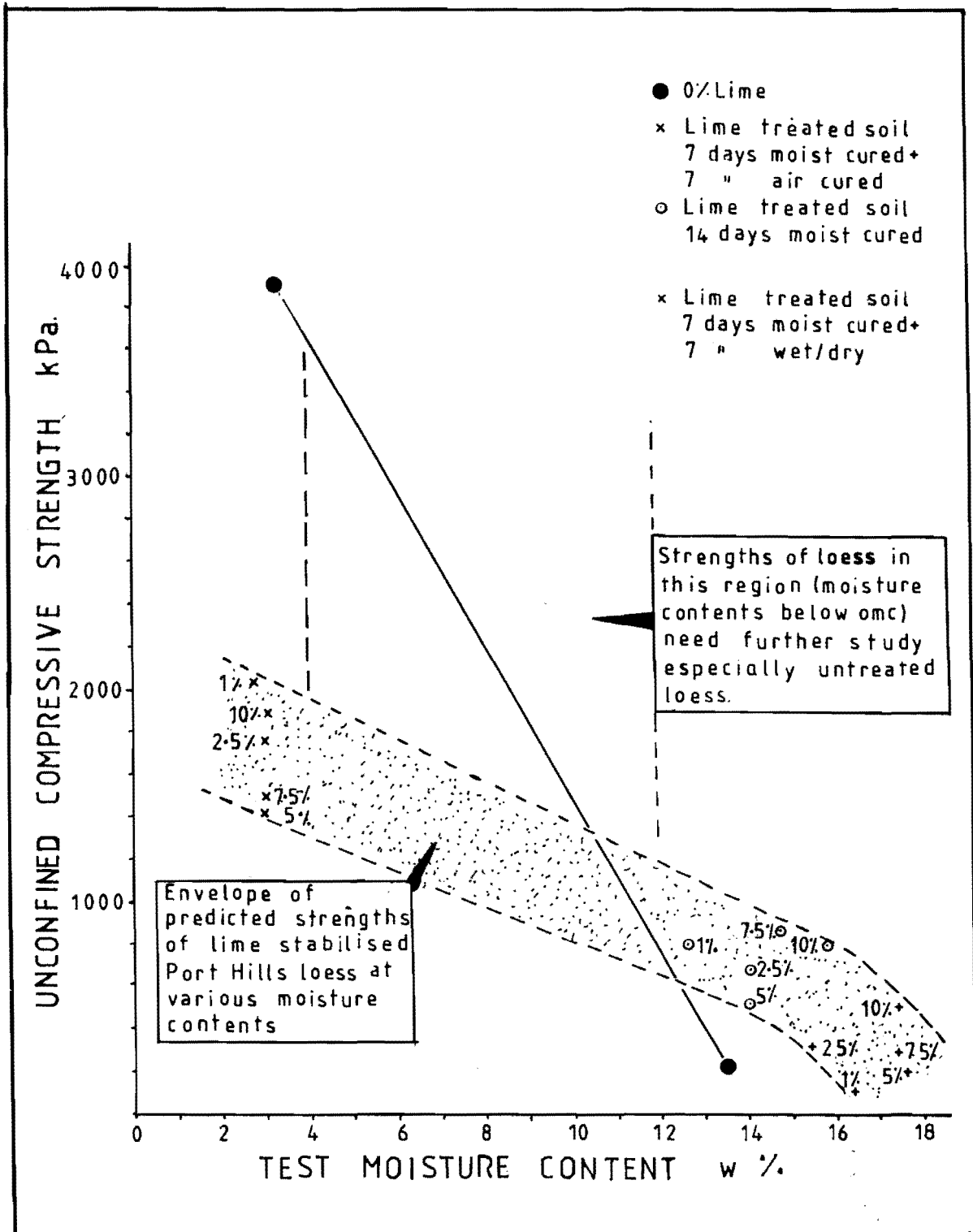


Fig. 4.15. Moisture content at time of testing for the untreated soil and lime treated soils for the three curing periods.

4.6 THE EFFECTS OF LIME ADDITION ON STRAIN DEFORMATION

4.6.1 Strain deformation

Neubauer and Thompson (1972) have shown that the addition of lime to a soil reduces the amount of strain deformation (see fig 4.16.). They found that at 6% lime, strain deformation was reduced by a factor of 4. Figures 4.17 - 4.19 show the effects of increasing lime addition on strain deformation for the soil tested in this study and the results are discussed below.

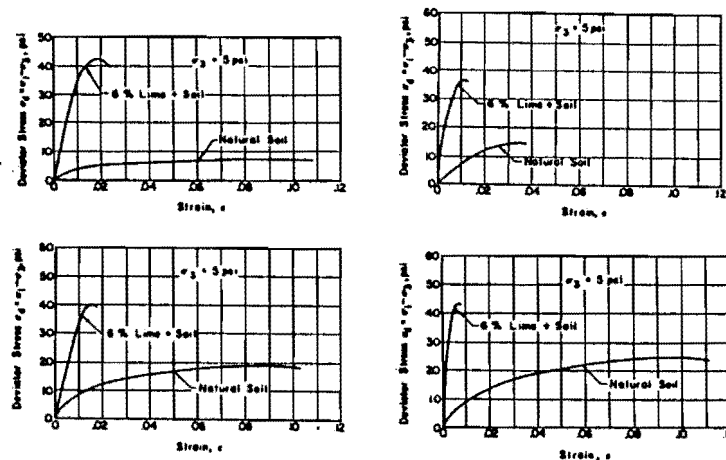


Fig. 4.16. The immediate effects of stress strain characteristics of four different soils treated with 6% lime (Neubauer and Thompson, 1972).

(i) Moist cured samples

Figure 4.17 shows axial strain deformation for various additions of lime for the moist cured samples. The immediate effect of lime (at 1% and 2.5%) is to reduce strain deformation by a substantial degree. The addition of 5% lime to the soil slightly increases strain deformation and lime percentages above this have little further effect.

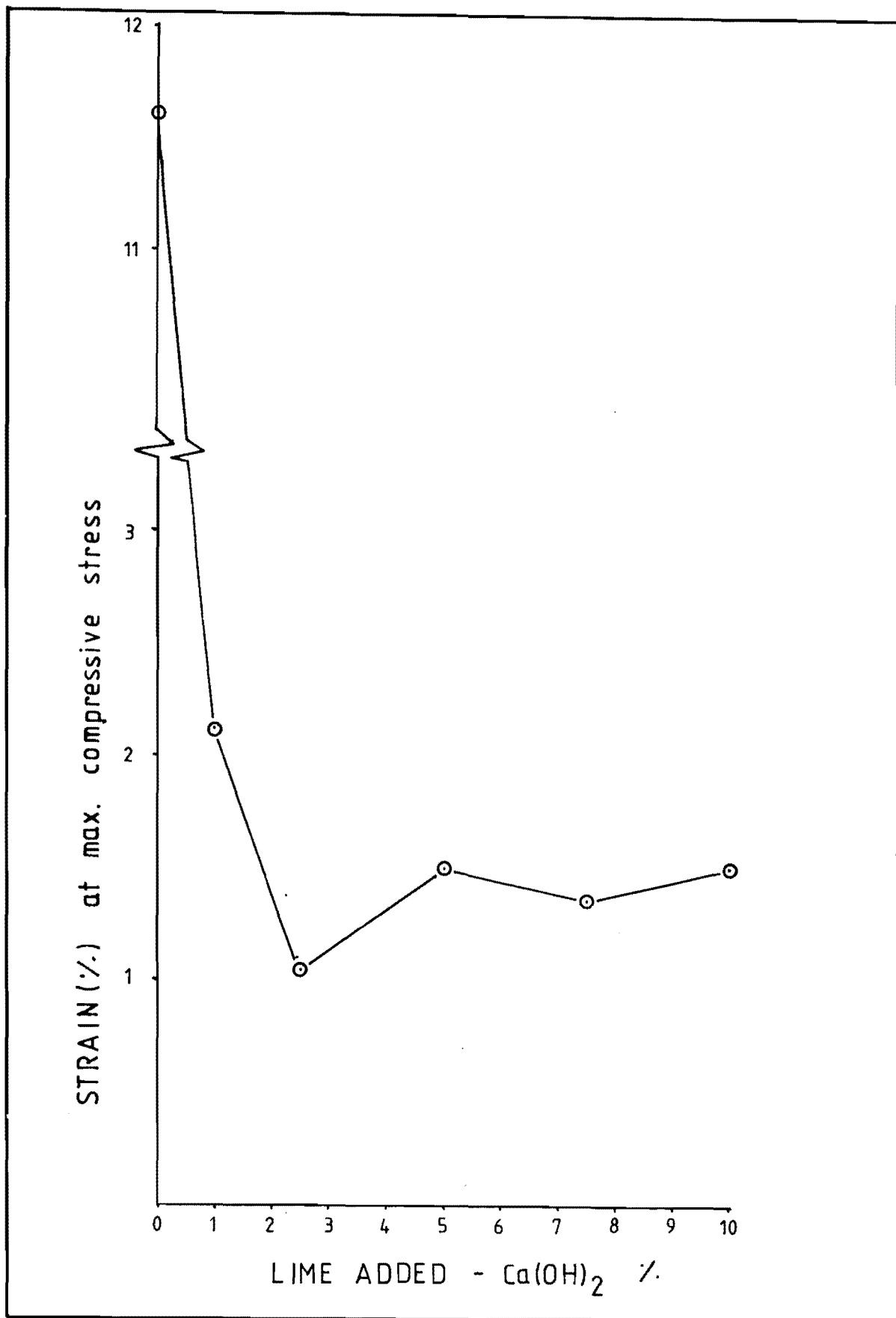


Fig. 4.17. The effects of lime addition on strain deformation for those samples moist cured 14 days. Strain deformation is considerably reduced by the addition of small percentages of lime (1% - 2.5%).

(ii) Air dried samples

A similar trend to that of the moist cured samples is displayed by the air dried samples (see fig. 4.18). That is, an initial reduction in strain, followed by a slight increase at 5% and little further effect with higher percentages of lime. However the untreated air dried soil behaved as a brittle material (refer to fig. 4.21) and the initial reduction of strain with the addition of lime is minimal.

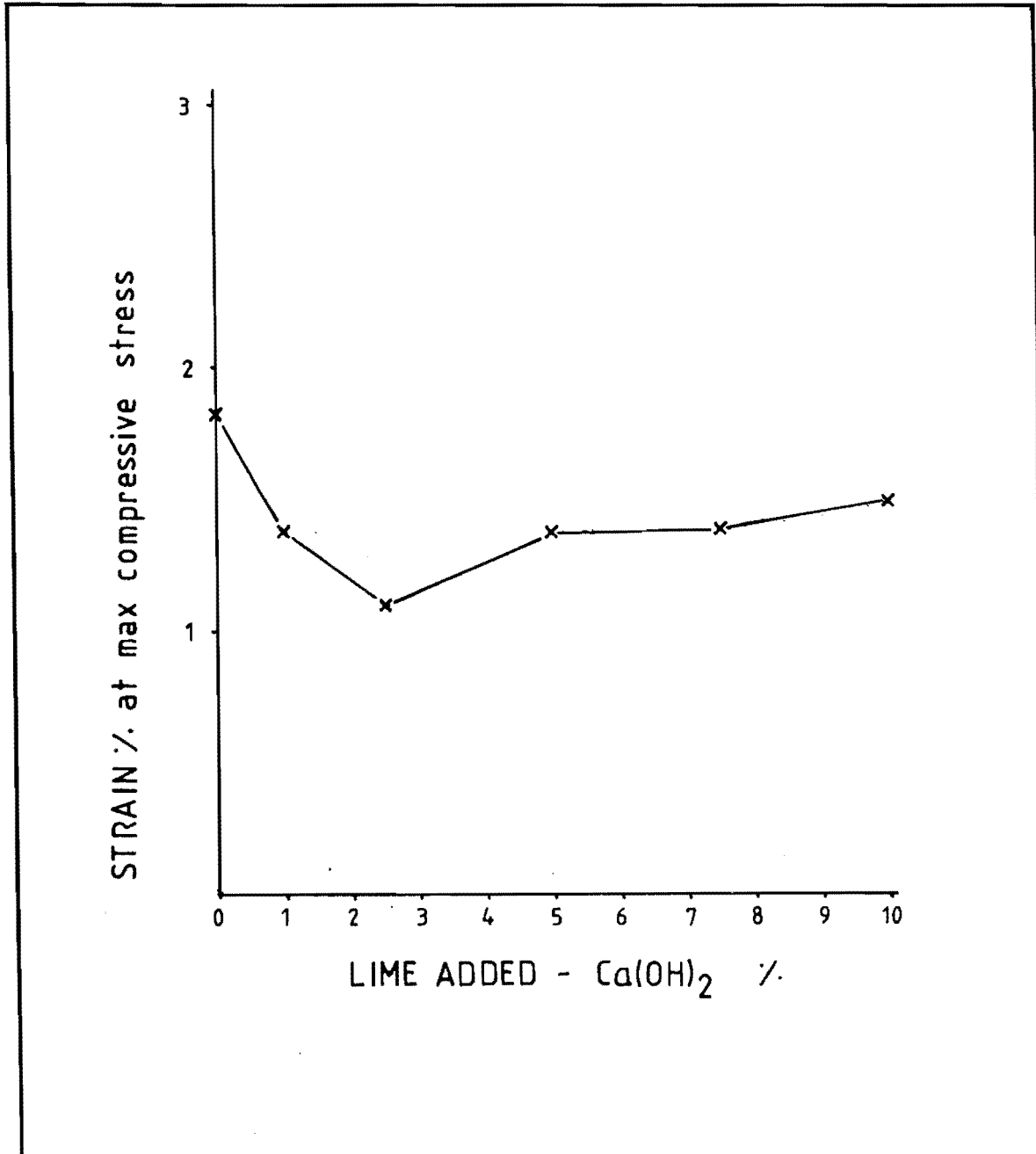


Fig. 4.18. The effects of lime addition on strain deformation for those samples 7 days moist cured + 7 days air dried.

(iii) Soaked samples

As with the air - dried and moist cured samples, there is an initial reduction of strain the addition of 1% and 2.5% lime for the soaked samples (see fig. 4.19). Unlike the other two curing methods, strain is further reduced with the addition of 5% lime although the compressive strength is decreased. Strain deformation is increased with higher percentages of lime, and is greatest with the addition of 10% lime which has the highest compressive strength.

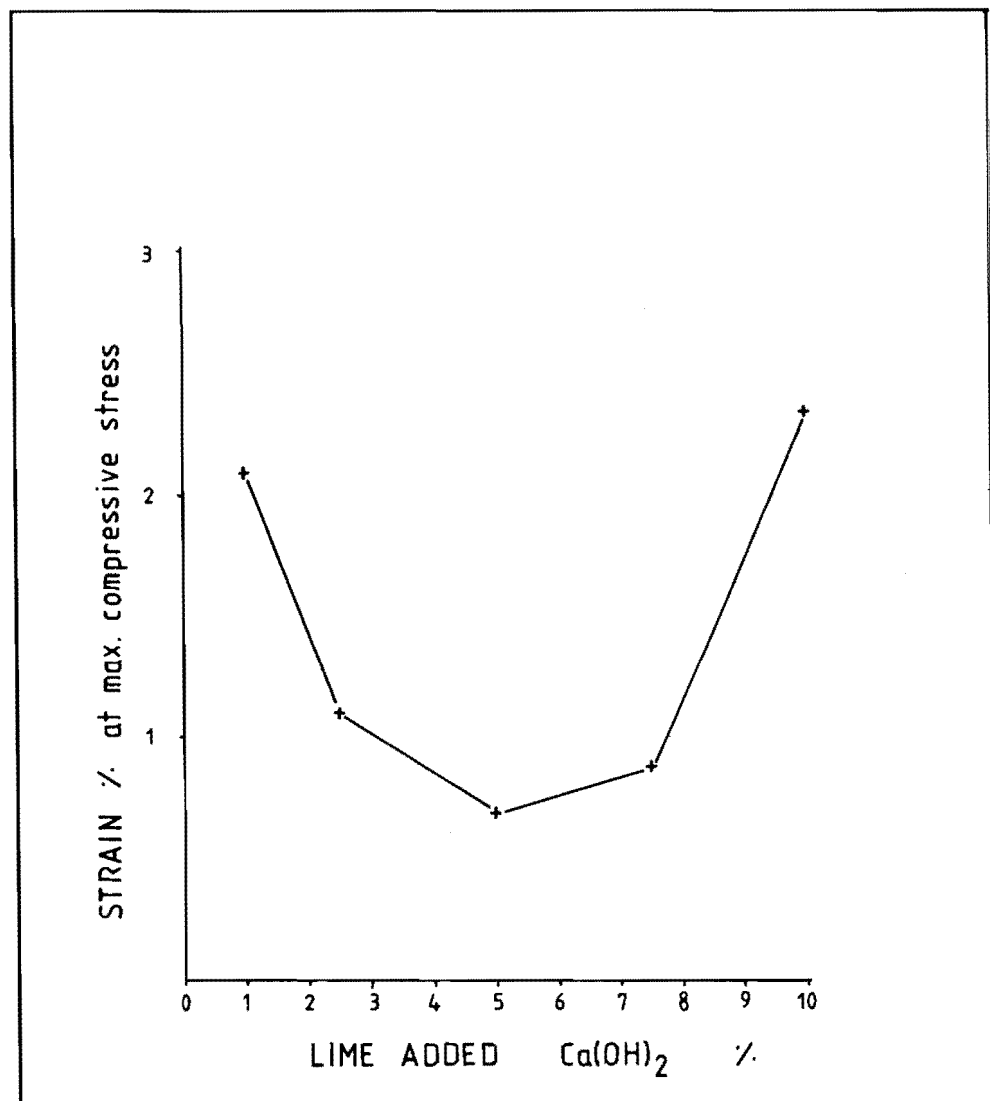


Fig. 4.19. The effects of lime addition on strain deformation for those samples 7 days moist cured + 7 days wet/dry. The untreated soil did not survive the wetting and drying process and no data was available.

4.6.2 Stress / strain relationships

Figure 4.20 shows the effects of lime treatment on the modulus of elasticity as reported by Neubauer and Thompson (1972). The diagram shows that the modulus of deformation is increased by a factor of 10 at 6% lime for some soils. The modulus of deformation peaked at 4% lime for one soil. Brandl (1981) reported that the modulus of elasticity initially increased with the addition of 1% lime (20 - 40 times) and that the lime stabilised soil acted as a "brittle" material. Further additions of lime reduced the modulus of deformation. Croft (1964) reported that lime stabilised clay can be regarded as a brittle material.

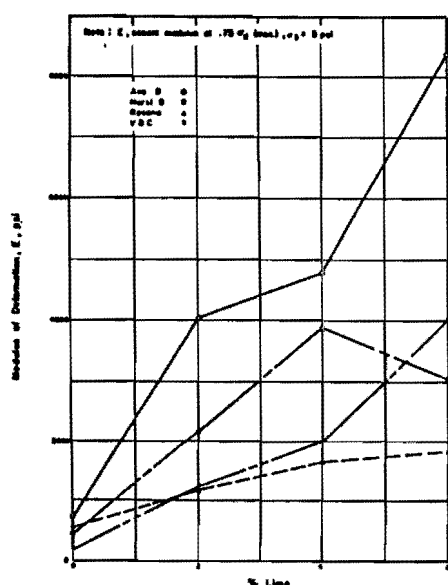


Fig. 4.20 The immediate effects of lime treatment on Modulus of deformation (E) (Neubauer and Thompson, 1972).

Figure 4.21 shows the effect of increasing lime addition on the modulus of elasticity ($E_{(50)}$ - tangent method, Brown, 1980) for the soil tested in this study and the results are discussed below.

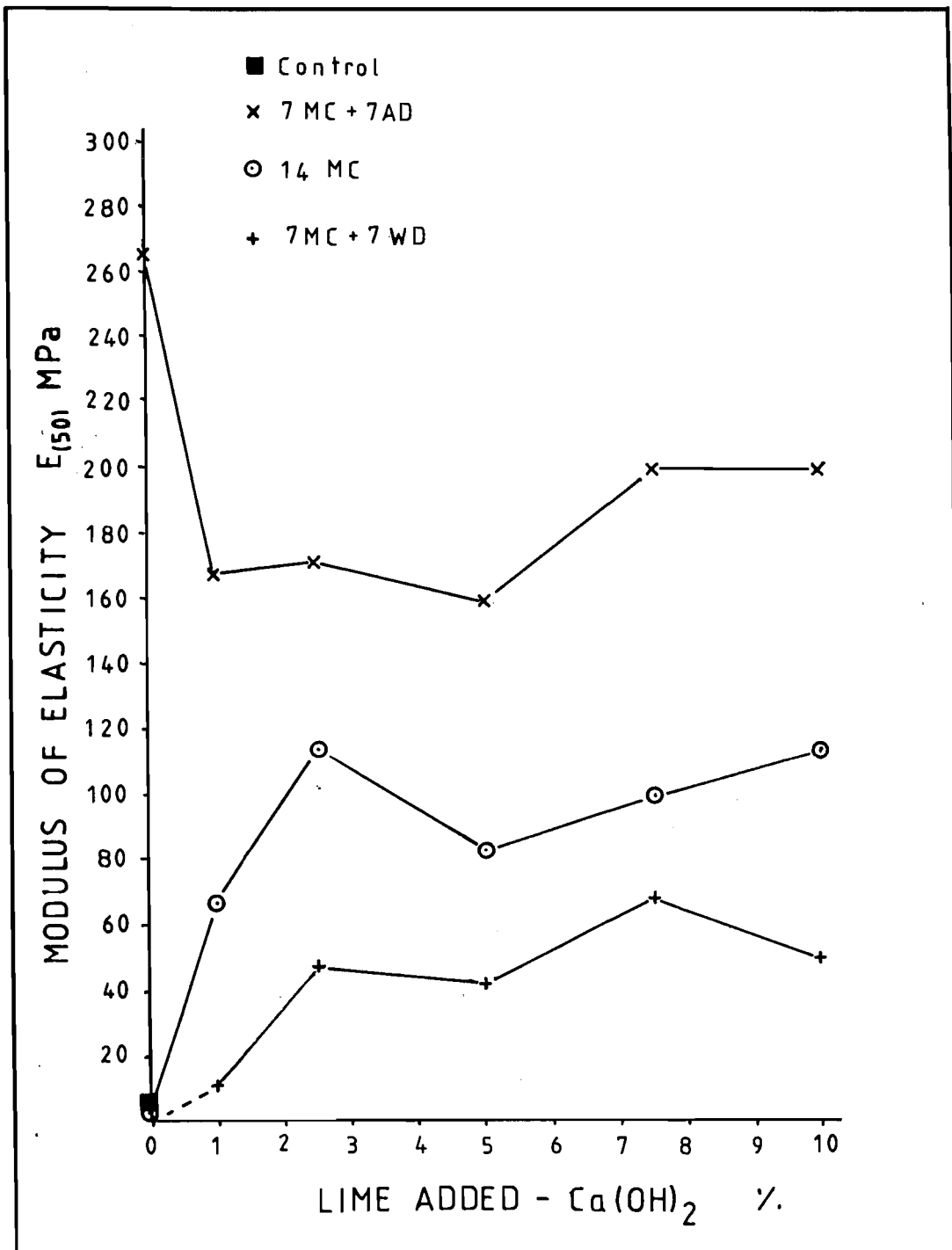


Fig. 4.21. The Modulus of Elasticity ($E_{(50)}$ - tangent method) for the control sample, the untreated soil and lime treated soil for the three different curing methods.

(i) Moist cured samples

The untreated, moist cured sample has a modulus of deformation (2-3 MPa), typical of silts as given by Lee, White and Ingles (1983). The addition of 2.5% lime has increases the modulus by approximately 60 times that of the untreated soil. There is decrease in the modulus at 5% lime, followed by an increase with 7.5% and 10% lime. Data show that the modulus of elasticity is raised, due to the increases in compressive strength and reductions in strain deformation at 1% and 2.5% lime. The lowering of the modulus at 5% lime is due to the reduced compressive strength and increased strain. Although strain deformation is not futher reduced at 7.5% and 10% lime, increased compressive strengths raise the modulus of elasticity.

(ii) Air dried samples

The untreated air dried soil the highest modulus of elasticity (260 MPa) indicating a brittle deformation behaviour. 1% - 5% lime reduces the modulus and the addition of lime above 5% raises the modulus, which appears to peak at 7.5% lime. Unconfined compressive strength data indicate that while the strength of the lime treated soil is reduced at 1%, 2.5%, and 5% lime the soil still behaves as a brittle material due to the reduction of strain deformation. Strain deformation is not futher decreased with 7.5% and 10% lime, but the modulus of elasticity is raised due to increases in compressive strength.

(iii) Soaked samples

For the soaked samples, the increases in the modulus of elasticity with the add_tion of lime is not as great as those of the moist cured and air dried samples, due to lower compressive strengths. Unlike the other curing methods, strain is further reduced at 5%, accounting for the less marked reduction in the modulus of elasticity at this lime content. The modulus is increased at 7.5% lime due to increased compressive strength, but is reduced at 10% lime due to increased strain deformation.

4.7 SYNTHESIS

4.7.1 Strength gains

Table 4.2 summarises the strength gains of lime stabilised loess.

Table 4.2 Summary of strength gains of lime stabilised loess.

	Control strength kPa	Lime % strength optimised	Maximum strength kPa	Strength gain
14 days moist cured	145	7.5	870	6x
7 days moist cured + 7 days air dried	145	10% +	1960	27x
7 days moist cured + 7 days wet/dry	145	10% +	480	4x

(i) The untreated soil, moist cured for 7 days, followed by 7 days air drying, gave the highest unconfined compressive strength, 20 times the strength of the untreated 14 day moist cured sample, and twice that of the soil treated with up to 10% lime (see fig. 4.22).

(ii) The greatest strength gains on the addition of lime are obtained for air dried samples. Strengths are between 2 and 3 times greater than those moist cured, lime treated samples; and more than 10 times the strength of the uncured untreated soil (control). A combination of initial moist curing followed by a period of air drying gives greatest strength gains (see fig 4.22).

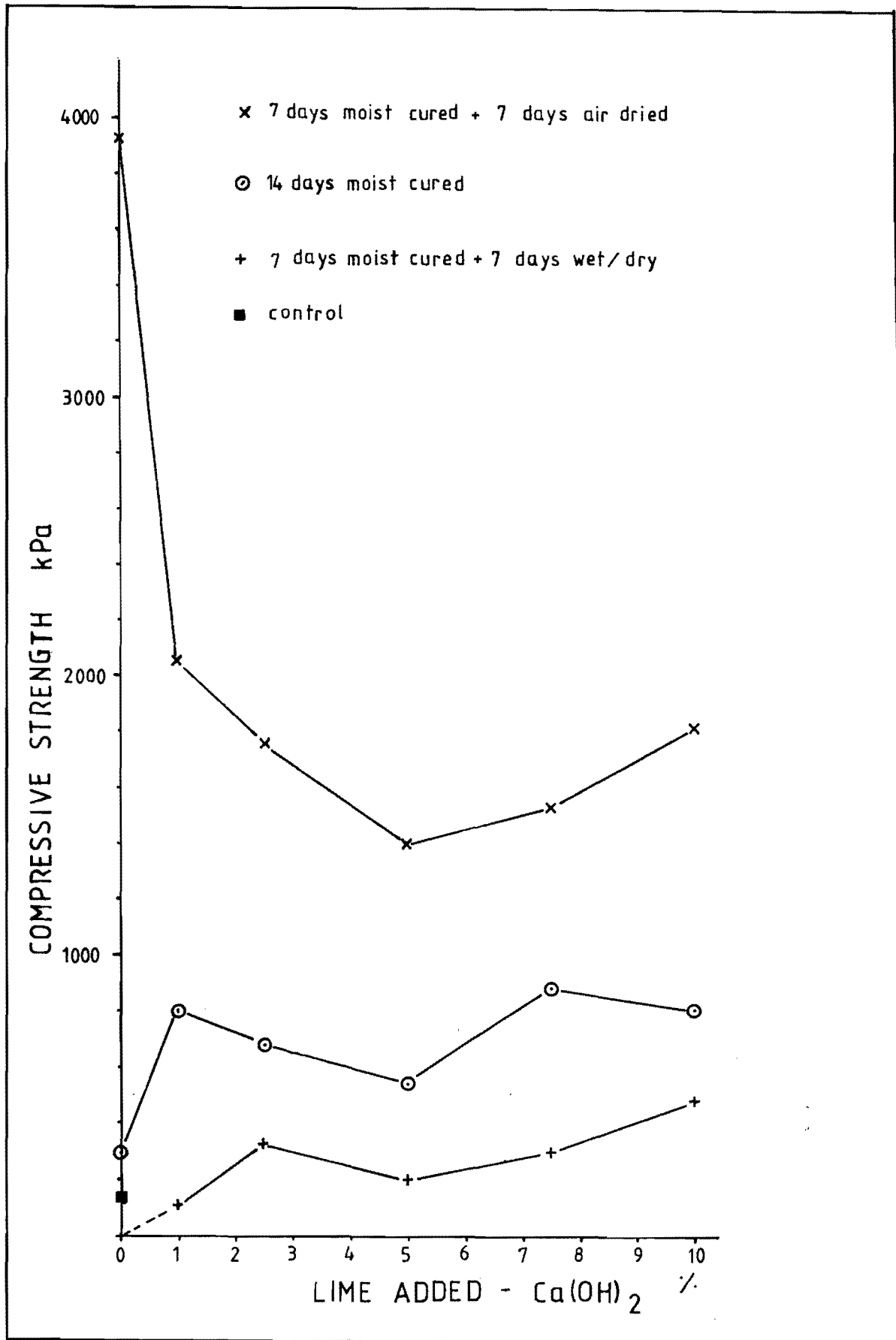


Fig. 4.22. The effects of lime addition on the unconfined compressive strength for the three curing methods.

(iii) For moist cured and soaked samples, the maximum compressive strength of the lime treated soil is between 2 and 3 times greater than the strength of the untreated soil (see fig 4.22).

(iv) There appears to be an initial strength gain of the soil with the addition of up to 2.5% lime. This is followed by a decrease in strength gains with the addition of between 2.5% and 5% lime (see fig. 4.22).

(v) The amount of lime to give optimum strength gains for the soil tested appears to be 7.5% for moist cured samples, and above 10% for air dried and soaked samples (see fig. 4.22).

4.7.2 Strain deformation

(i) The addition of lime to the soil reduces strain deformation. Strain deformation is minimised with the addition of between 2% and 5% lime, depending on the curing methods. For the moist cured and air dried samples, the addition of more than 5% lime did not further reduce strain. For the soaked samples the addition of lime above 5% increased strain.

(ii) Minimum strain deformation does not correlate to maximum compressive strength obtained on the addition of lime for any curing method.

(iii) It is apparent from the data, that lime stabilised soils perform in a brittle fashion, undergoing small amounts of deformation at relatively low compressive strengths (cf. rock materials).

4.7.3 Further investigations

(i) The untreated, air dried soil gave the highest compressive strength. This has been explained by the effects of soil suction at low moisture contents. Figure 4.15 shows the effect of moisture content at the time of testing on the compressive strengths. Further investigations should be carried out on the strengths of soils at moisture contents indicated in figure 4.15. The effects of soil suction on the strength of the untreated and lime stabilised soils

should also be carried out. Soil suction can be measured simply by psychrometer measurements, vapour methods, or pressure membrane methods as outlined by Winterkorn and Fang (1975) and Richards (1965).

(ii) There is a reduction in strength gains for lime treated soils between 2 and 5 percent lime addition. Reaction products formed at low lime percentages (1% - 5%) require further investigation, using analytical chemical methods, X-ray diffraction methods and Scanning Electron Microscope methods.

(iii) The effects of carbonation of hydrated lime by atmospheric CO should be investigated further. Carbonation should be completely excluded during the curing periods, and the amount of carbonated lime compared for these samples and samples exposed to the atmosphere and completely immersed during curing.

(iv) Research (Laguros et al. 1956, Herrin and Mitchell, 1961, Remus and Davidson 1961, Croft, 1964, and Machan et al. 1977) has indicated that a minimum curing period of approximately 28 days is required to give lime stabilised soils effective erosion resistance and strength development. Further investigation into the effects of extended curing periods on the strength characteristics of lime stabilised loess is required.

CHAPTER FIVE

SUMMARY AND CONCLUSIONS

5.1. LOESS AND EROSION

(i) The loessial soils of the Port Hills are susceptible to erosion due to the erosive and dispersive nature of the soils. Tunnel gully formation is a major erosion hazard.

(ii) Special attention during planning must be taken to define the erosion problems that may be encountered during development on the Port Hills. The soil must be characterised, possible erosion hazards identified, and remedial measures considered, to reduce or prevent further erosion. The production of site development models and erosion models are useful techniques.

5.2. CHARACTERISATION OF LOESS FOR LIME STABILISATION

Lime stabilisation is one method that can be used to prevent and reduce erosion hazards. However before lime stabilisation can be used the geotechnical properties of the soil must be ascertained to:

- 1) characterise the soil and identify the erosion hazard. Pinhole erodibility, Emerson dispersion, swelling strain, and plasticity indices are useful test methods for such characterisation;
- 2) determine whether the soil is suitable for lime stabilisation. Although there is no one test that can determine this, there are a number of soil property changes that can occur on the addition of lime and these should be ascertained before lime is used as a stabiliser. Soil properties that are important in recognising suitability for lime stabilisation are, grain size distribution to determine the amount of clay material in the soil, clay mineralogy to predict the likely reaction of lime with the soil, and plasticity to determine the likely reactivity of

- the soil with lime;
- 3) determine the effects of lime stabilisation on soil properties, such as changes to optimum moisture content and maximum dry density, the reduction of erosion and swelling potential, and likely strength gains that may be obtained;
 - 4) determine the optimum and economic proportions of lime needed to provide maximum erosion resistance and / or maximum strength gains.

5.3. SOIL PROPERTIES OF LIME STABILISED LOESS

(i) Increasing lime content increases the optimum moisture content, and reduces the maximum dry density on compaction.

(ii) Small amounts of lime (1% - 2.5%) added to the loessial soil tested alters soil properties sufficiently to produce a non-erodible, non-dispersive soil.

(iii) Swelling potential and slaking are significantly reduced with the addition of small percentages of lime (1% - 2.5%), but higher percentages of lime are required to minimise swelling (7.5%) and the effects of slaking (5%).

(iv) Plasticity is increased on the addition of up to 5% lime due to an increase in the plastic and liquid limit, and is reduced with the addition of 7.5% lime, but not below that of the untreated soil.

(v) The overall increase in grain size of the soil, from a clayey silt to a sandy silt, on the addition of lime, makes for a more workable soil.

5.4. STRENGTH CHARACTERISTICS OF LIME STABILISED LOESS

5.4.1 Strength gains

(i) Air dried strengths of loess are above that of lime stabilised loess cured in the same manner, although the saturated loess has nil compressive strength.

(ii) The strength of loess is increased by 3 to 14 times with the addition of lime, depending on the curing method.

(iii) Greatest strength gains are obtained for those samples moist cured for 7 days followed by 7 days air drying, although for the same curing period the addition of lime reduced the strength of the untreated soil.

(iv) Strength gains of loess are optimised at 7.5% lime addition for samples moist cured for 14 days. The optimum lime content to give maximum strength gains for the air dried and soaked samples appears to be greater than 10%.

(v) Although strength gains are obtained on the addition of lime, there is generally an initial increase in strength (with 1% lime) followed by a reduction in strength gain with the addition of between 2.5% and 5% lime. It would appear that strength gains are affected by the slow reaction of lime to the clay minerals (illite).

5.4.2 Strain deformation

(i) Strain deformation is reduced with the addition of lime to the soil (up to 5%). Higher percentages of lime have little further effect on strain deformation, except for those soaked samples, where the addition of more than 5% lime increases strain deformation, although not above that of the untreated, uncured soil.

(ii) Lime stabilised loess acts as a brittle material, showing small amounts of deformation without acquiring high compressive strengths when compared to strengths of rock materials that act in the same manner.

5.5 FURTHER INVESTIGATIONS

(i) Further research is required into the reactions and reaction products of lime at low lime percentages (1% - 5%) to account for the reduction of strength between 2.5% and 5% lime, and grainsize changes and plasticity increases at low lime percentages (1% - 5%)

(ii) Further investigations are required into the effects of soil suction on the high dry strengths of loess.

(iii) The contribution of carbonation of lime on strength gains of loess requires further investigation.

(iv) The effects of extended curing periods on strength gains of lime stabilised loess requires further research.

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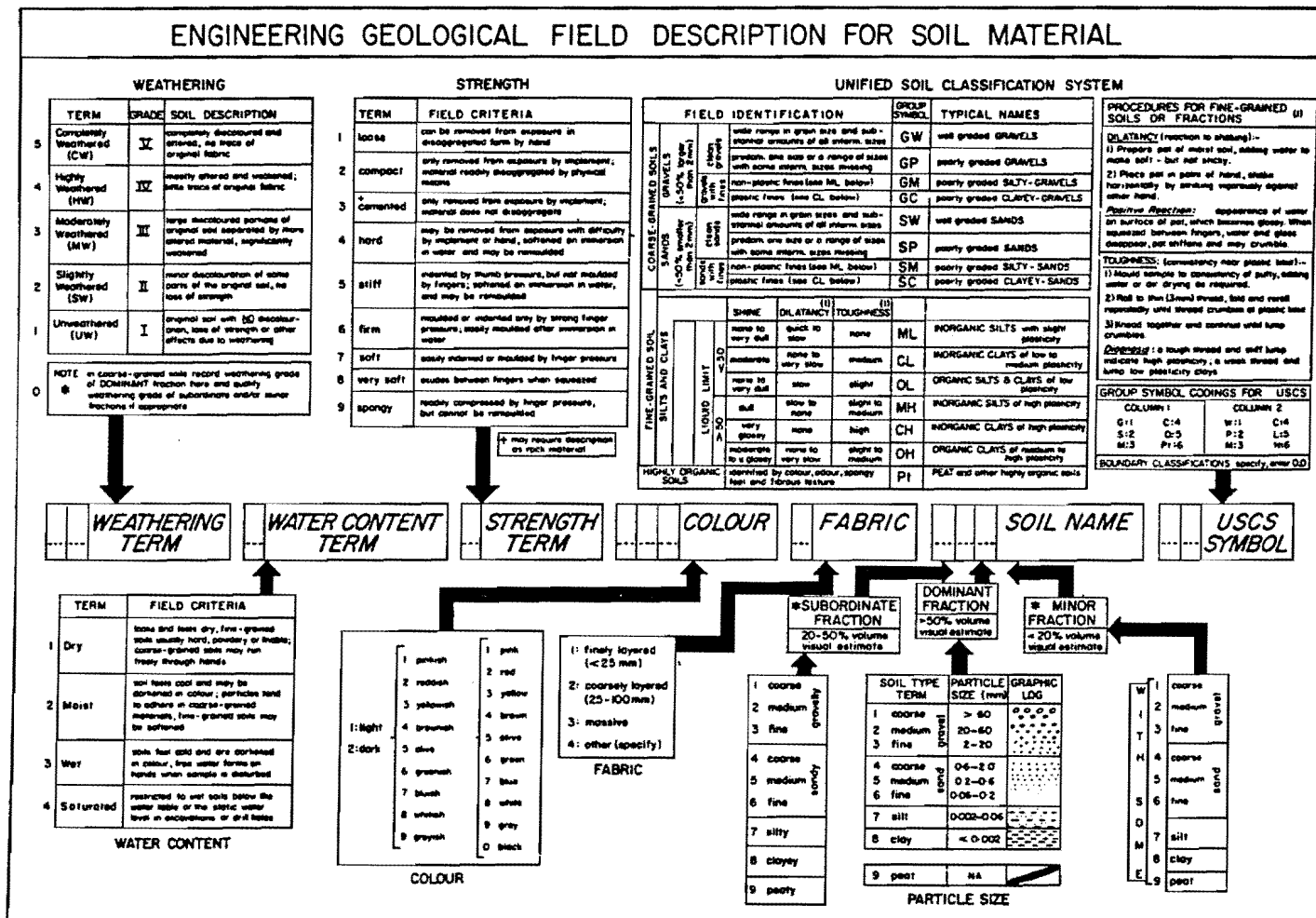
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APPENDIX 1

SOIL CLASSIFICATION

A 1. Soil material classification

A1.1. Soil material classification used for soil descriptions in this study (Bell and Pettinga, 1984).



APPENDIX 2

STRESS / STRAIN TESTING

A2.1. Introduction

The strength of lime stabilised loess was tested using the unconfined compressive strength. The unconfined compressive strength was tested because of the relatively simplicity of the test, the large number of samples tested, and because it is most commonly used to test strengths of lime stabilised soils.

A2.2. Procedure

Unconfined compressive strength of untreated and lime treated moulds (115mm length by 105mm diameter) was tested using a Wykeham - Farrance, 10000 kg stepless loading frame. An appropriate proving ring measured force (N) and axial strain deformation was measured using a dial gauge (see fig. A.2.1). Moulds were loaded at a rate of 1mm per minute and force measurements were taken every 0.1 mm of strain. Stress (kPa) was calculated from the force measurement (N) divided by the surface area (A , m^2) of each mould. Axial strain deformation is expressed as a percentage change in length of the sample.

A 2.3 Curing methods

A set of moulds (untreated soil) were tested immediately after compaction to act as curing controls. Lime stabilised moulds and untreated moulds were tested after three different curing methods to determine their effects on strength and other soil properties tested. The curing methods are given below.

- 1) 14 days moist cured at 20°C and at 99% + 2% relative humidity (RH). The moulds were placed in plastic bags to prevent excess moisture affecting the soil. According to (Laguros et al. 1956), moist curing at relatively normal temperatures and high humidities is most likely to represent conditions of curing encountered in the field (especially in roading sub-bases) and this method was used

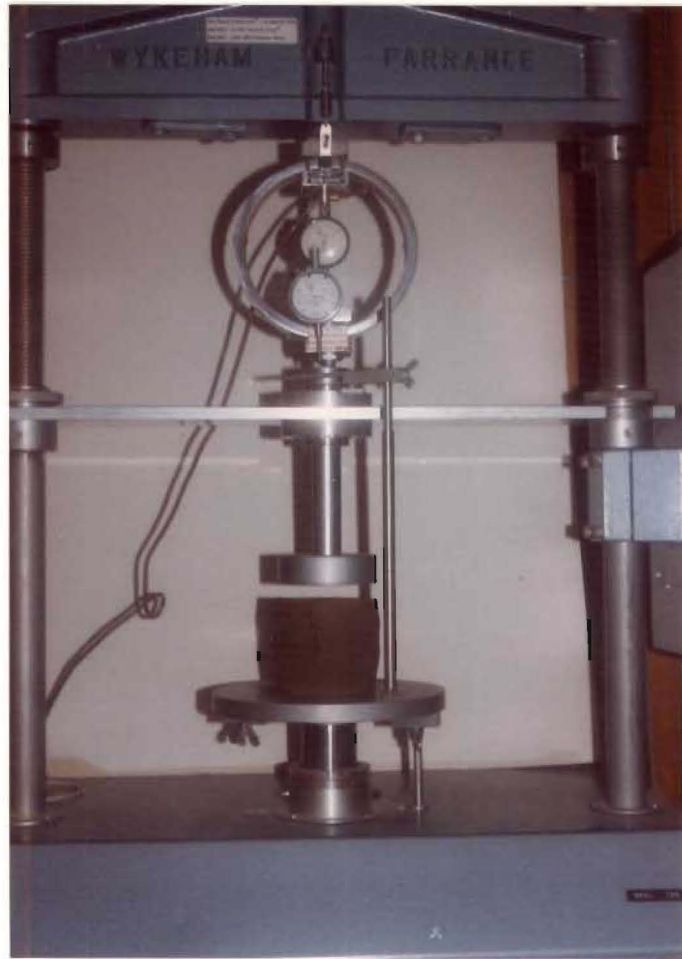


Fig. A.2.1. The Wykeham- Farrance 10 000 kg stepless loading frame used to measure compressive strength.

to simulate these conditions.

- 2) 7 days moist cured (as for 1) followed by 7 days air drying at room temperature ($20^{\circ}\text{C} + 5\%$). The placement of lime stabilised soils on the Port Hills often requires that the soil is exposed to the atmosphere and constant temperatures and humidities can not be maintained and this method was used to simulate these conditions.
- 3) 7 days moist cured (as for 1) followed by 7, 24 hour wetting and drying cycles. This method was used to determine the effects of slaking on lime stabilised loess and to determine the effects of cyclic wetting and drying on the strengths of the stabilised soils. The moulds were tested in the saturated state. Lime stabilised loess has been used for lining stormwater drains, watercourses and small artificial lakes (refer to Bell) and this method was used to simulate these conditions.

A 2.4 Strength data analysis

Five samples for each curing period at each lime percentage were tested. The mean stress at each 0.1mm strain was calculated and mean stress / strain curves plotted. Maximum and minimum values about the mean were also calculated and plotted as scatter envelopes.

Young's modulus of elasticity was calculated ($E_{(50)}$ - tangent method, Brown 1980) was calculated for the untreated samples and lime stabilised samples.

A 2.5 Results

The results of the stress / strain data acquired are discussed in chapter four (sections 4.4 - 4.6). Figures A.2.2. (a) - (c) show averaged stress / strain curves for the lime treated samples for the three curing methods. Figures A. 2.3 (a) - (c) show the averaged stress / strain curves and scatter about the mean for each lime percentage. The stress strain curves for the control samples and the untreated cured (0%) samples are discussed and given in chapter 4.4.

Young's modulus of elasticity results are given in figure 4.20 and discussed in section 4.6.2.

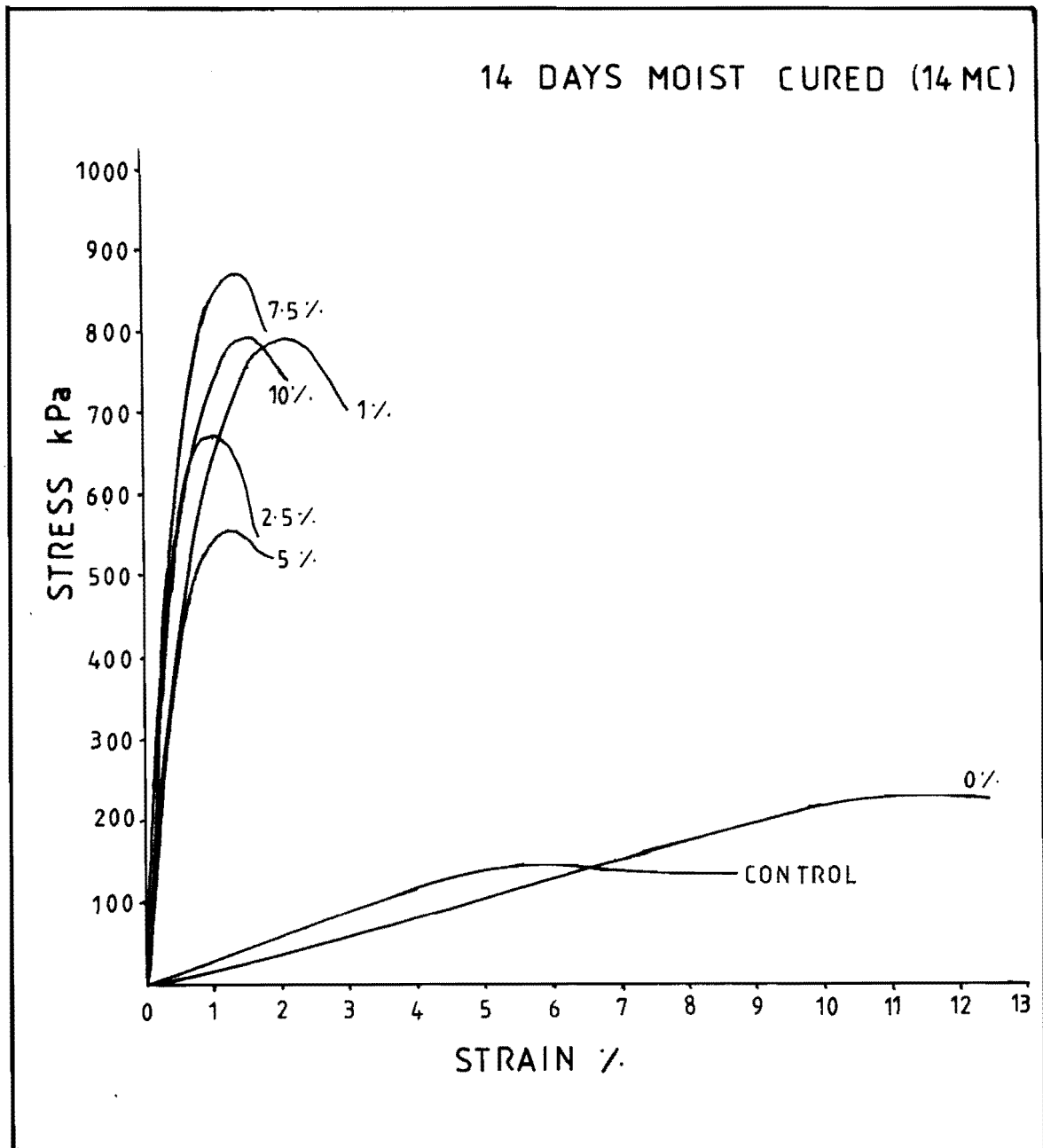


Fig. A.2.2(a) Stress/Strain curves for the untreated soil (0%) and lime treated soils moist cured 14 days at 20°C and 99%RH. Control tested immediately after compaction (i.e. no curing).

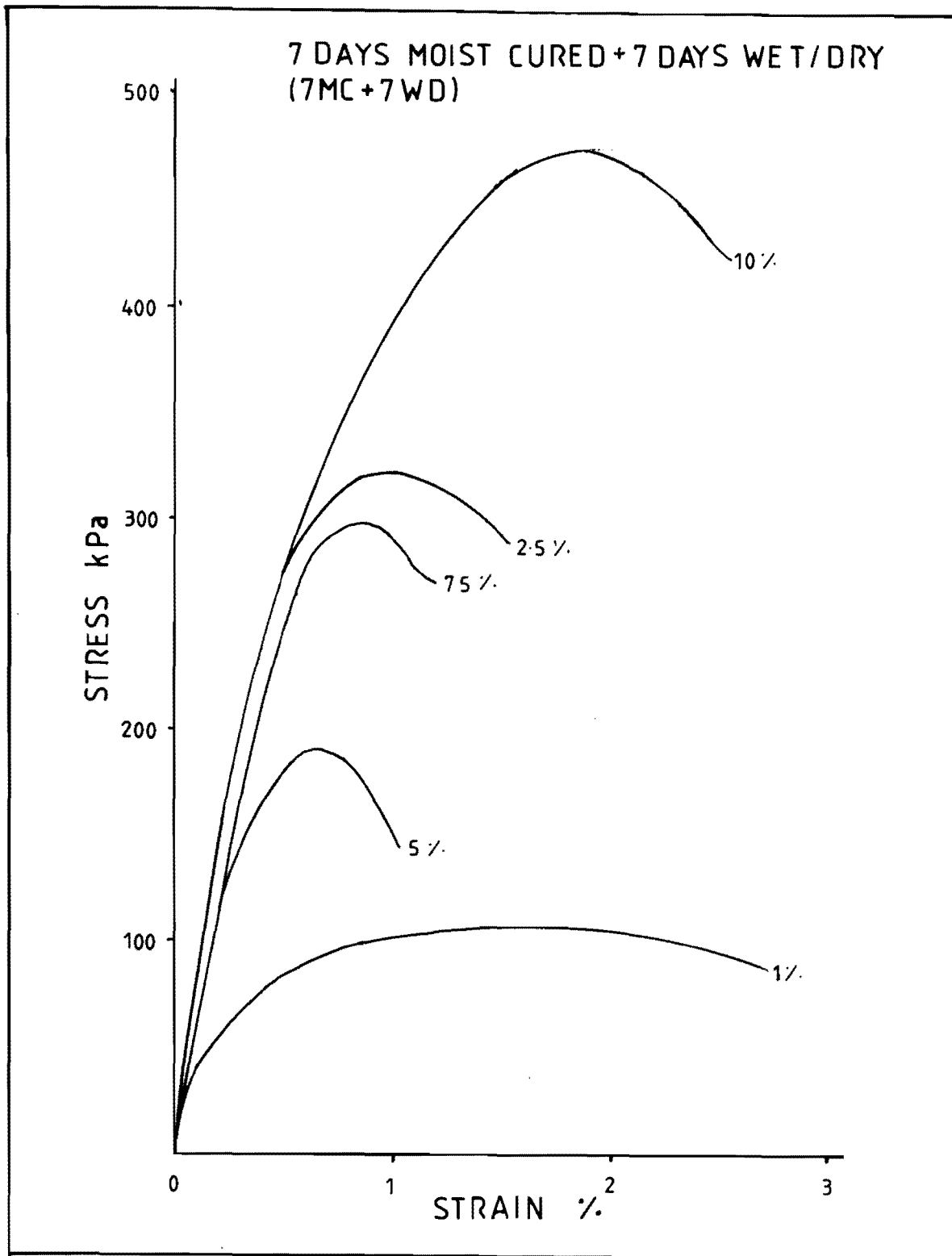


Fig. A.2.2(b) Stress/Strain curves for lime stabilised soil, moist cured 7 days, followed by 7 days of cyclic wetting and drying. Samples were tested in the soaked state.

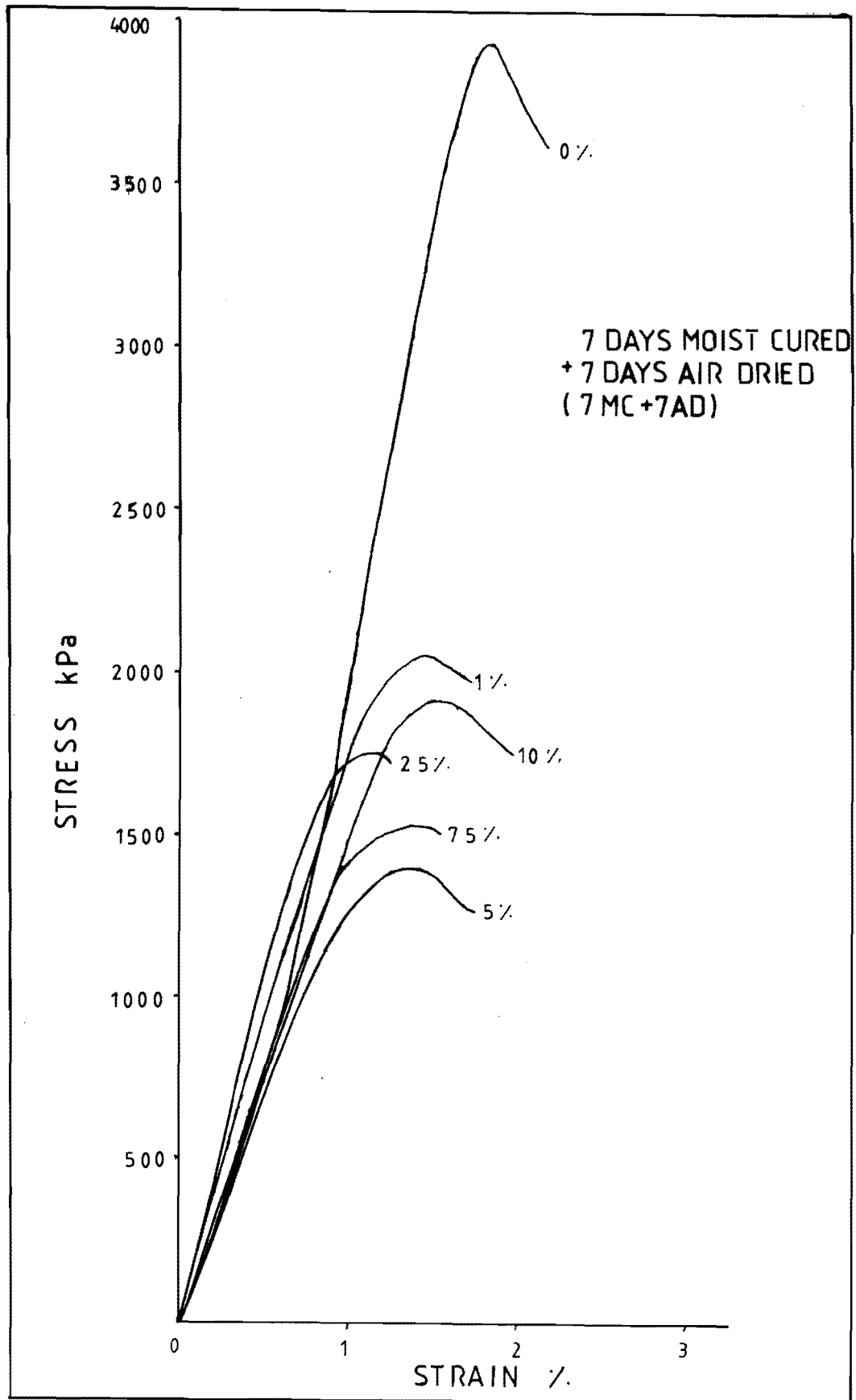


Fig. A.2.2(c) Stress/strain curves for the untreated soils (0%) and lime treated soils, moist cured for 7 days followed by 7 days air drying (at $\approx 20^{\circ}\text{C}$).

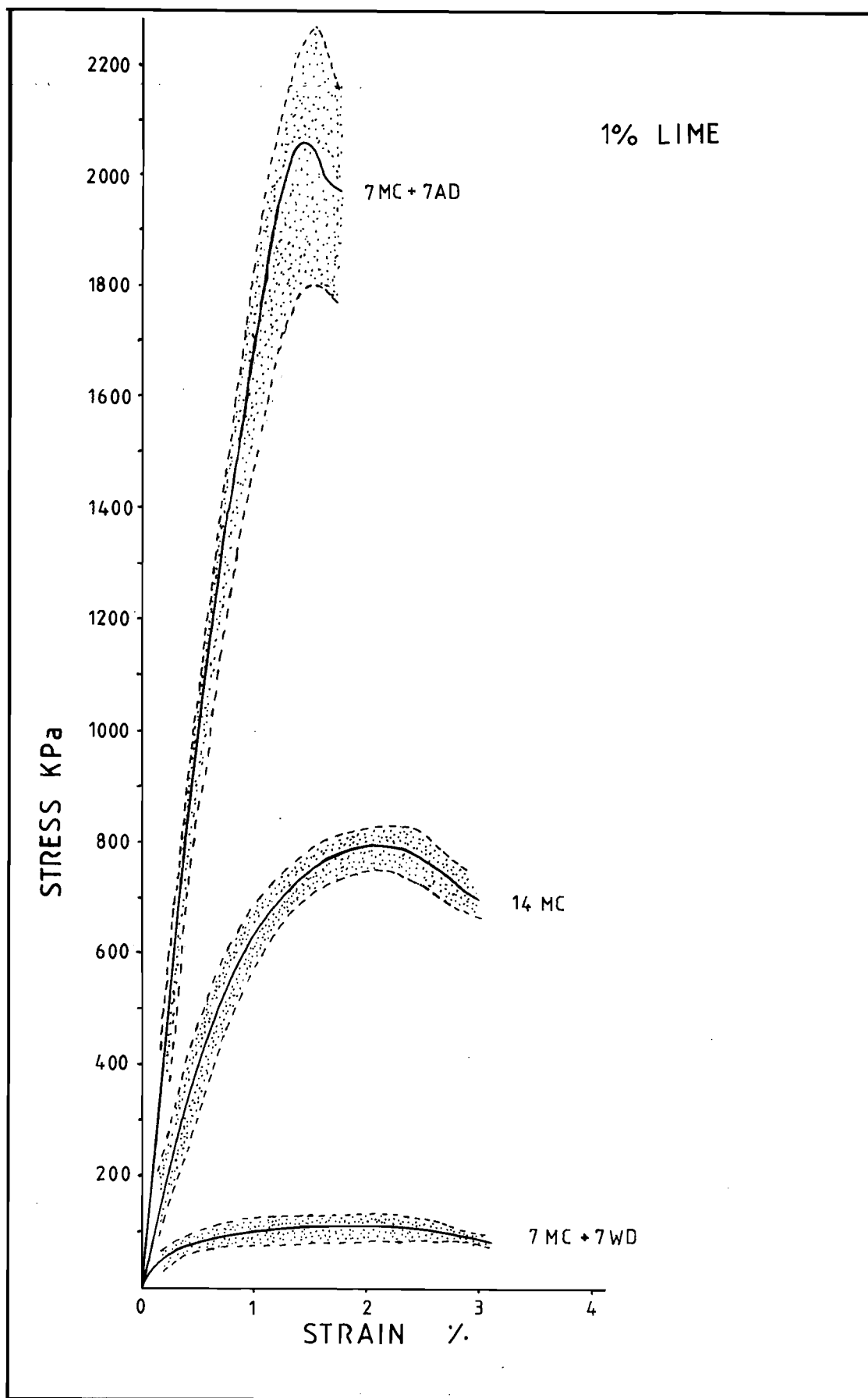


Fig. A.2.3(a) Stress/strain curves (and envelopes of scatter about the mean) of the soil treated with 1% lime for the three curing methods; 7 days moist cured + 7 days air dried (7MC + 7AD), 14 days moist cured (14MC), and 7 days wet/dry cycles (7MC + 7WD).

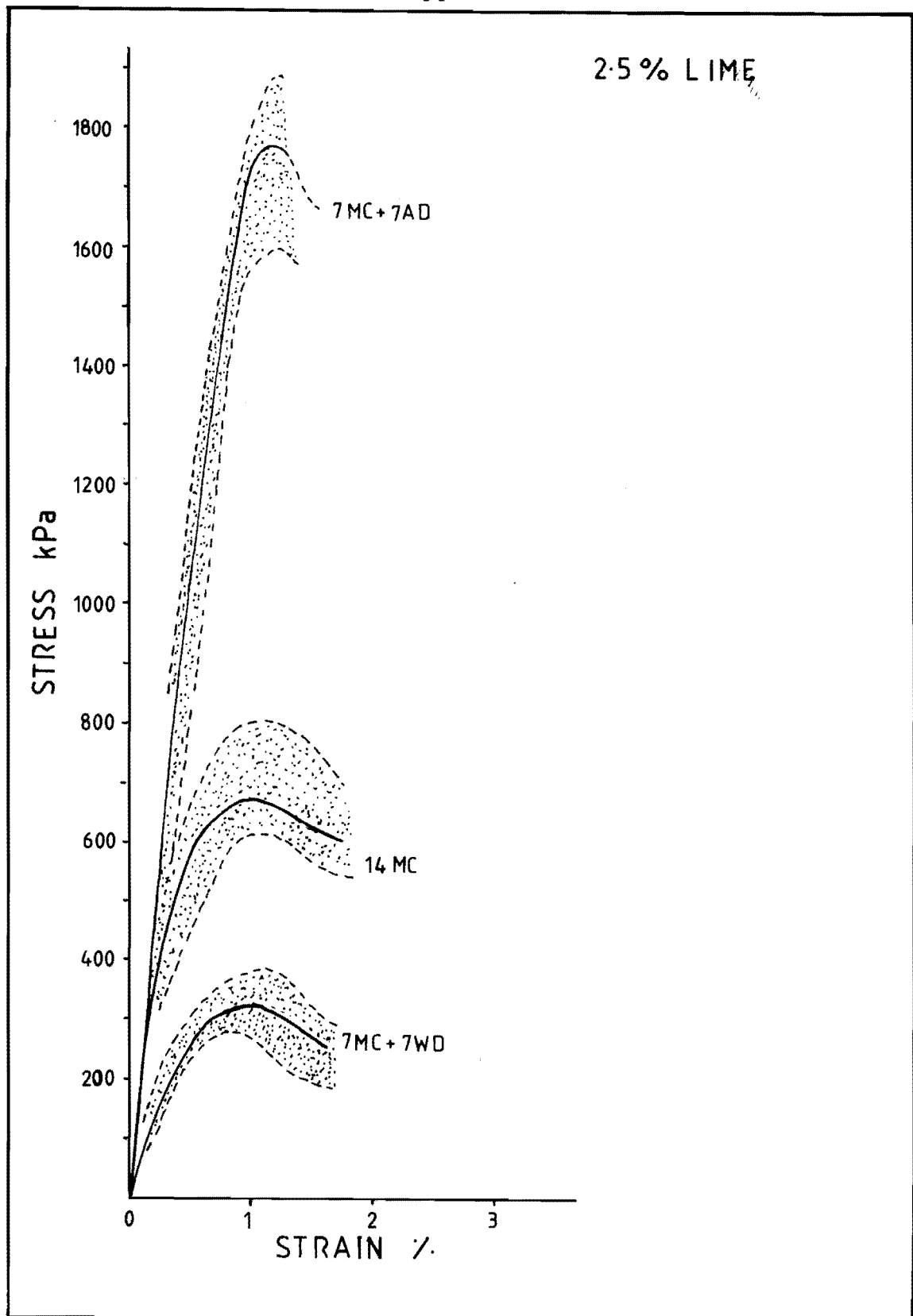


Fig. A.2.3(b) Stress/strain curves (and envelopes of scatter about the mean) of the soil treated with 2.5% lime for the three curing methods. (see Fig. A.2.3(a)).

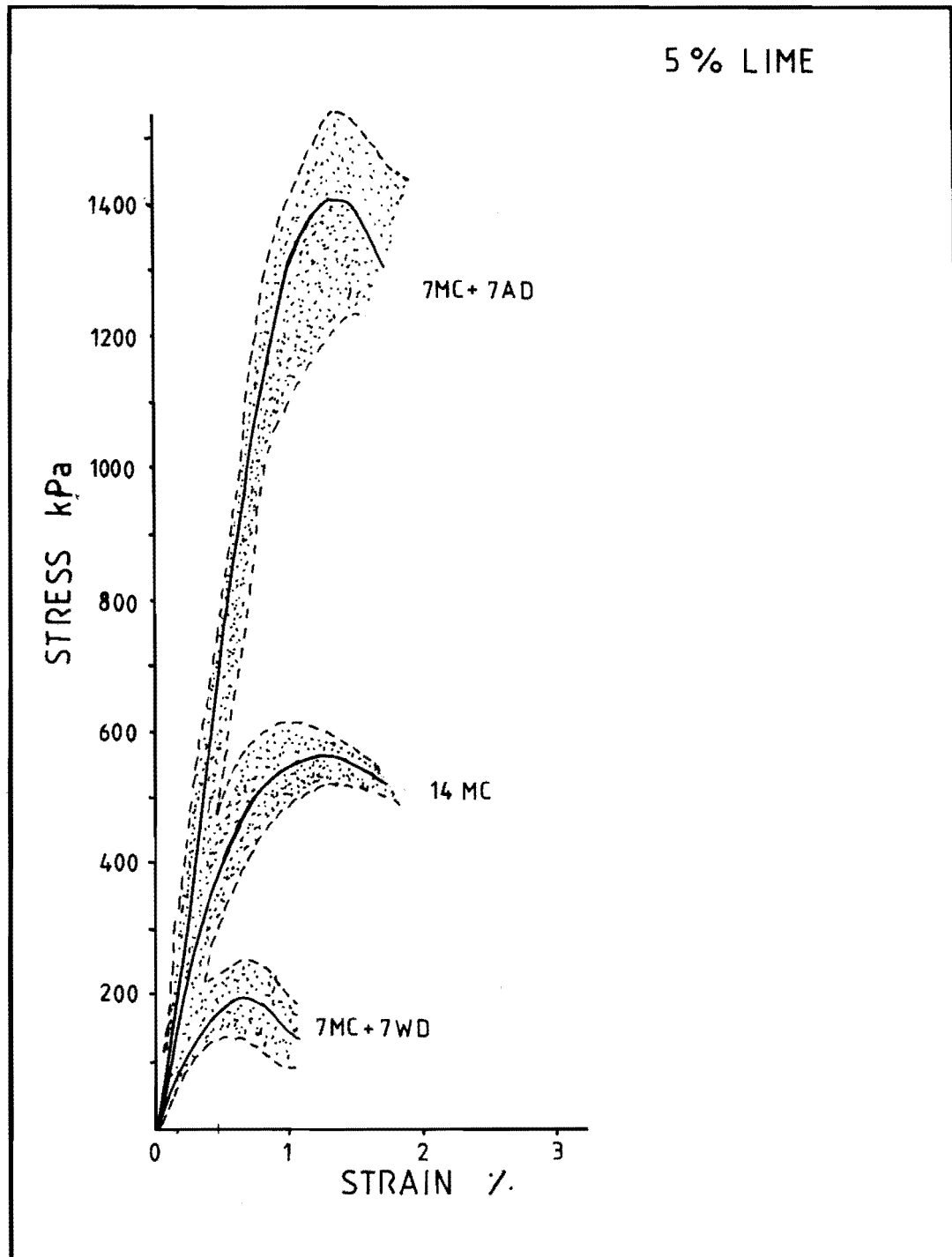


Fig. A.2.3(c) Stress/strain curves (and envelopes of scatter about the mean) of the soil treated with 5% lime for the three curing methods. (see Fig. A.2.3(a)).

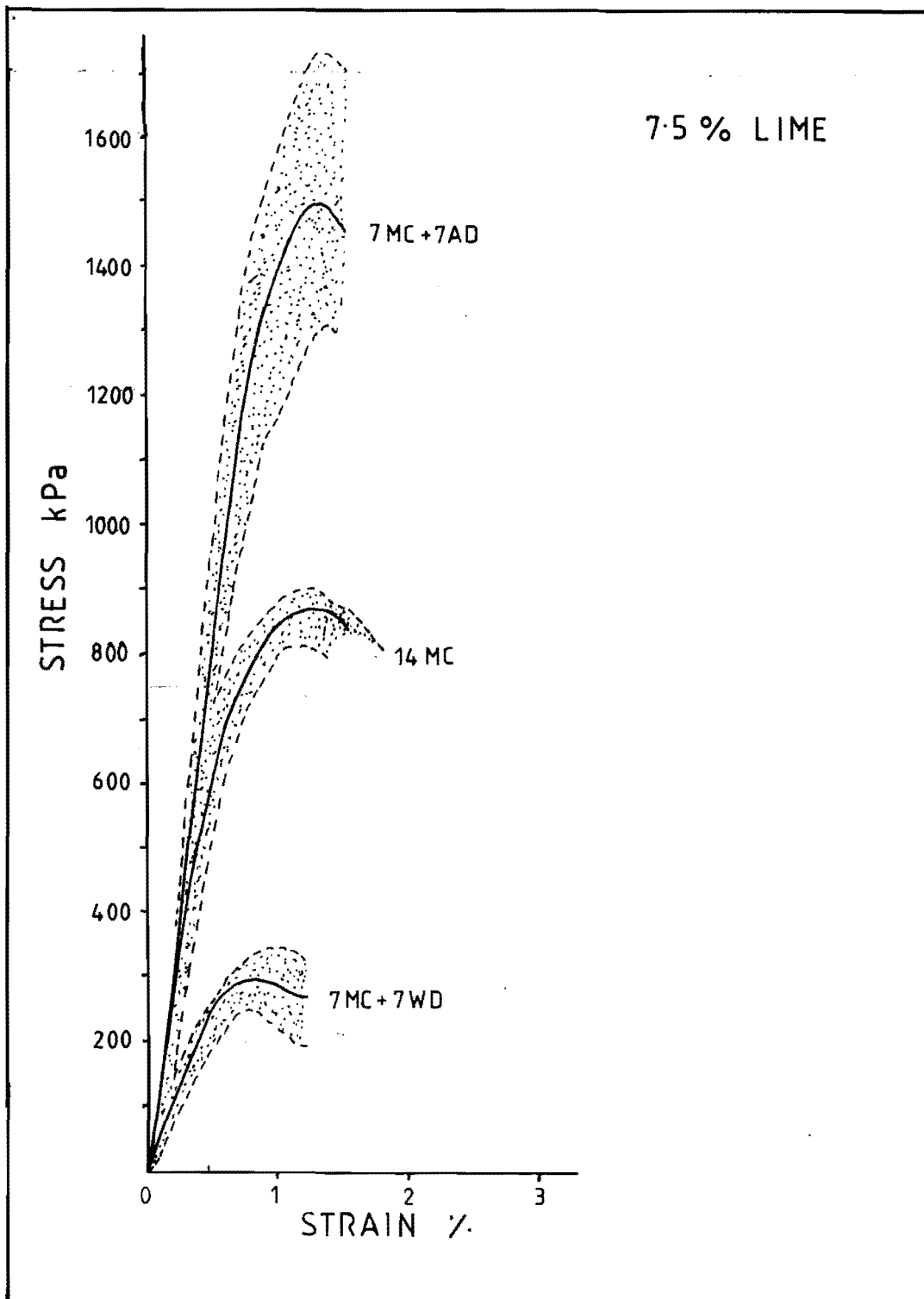


Fig. A.2.3(d) Stress/strain curves (and envelopes of scatter about the mean) of the soil treated with 7.5% lime for the three curing methods. (see Fig. A.2.3(a)).

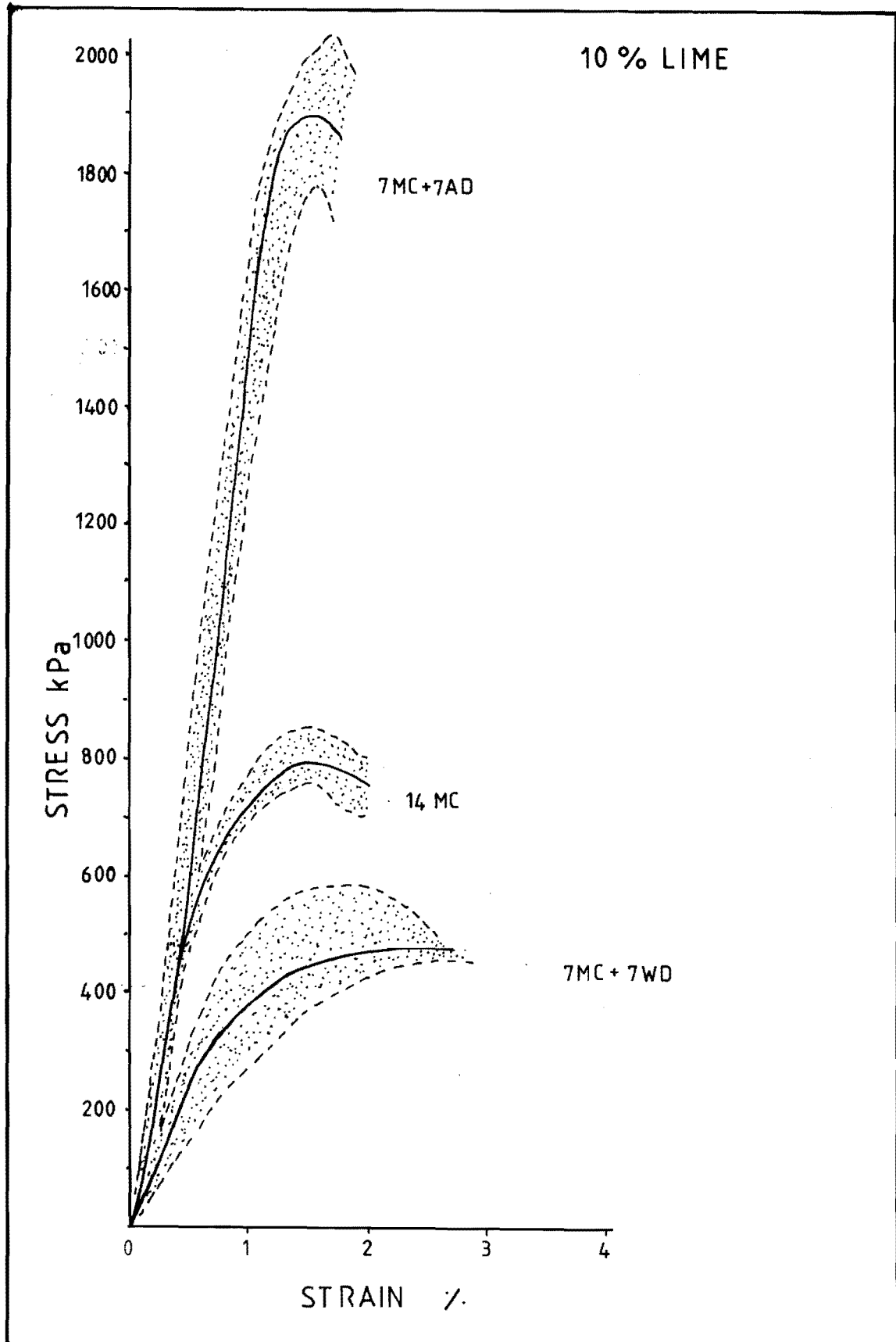


Fig. A.2.3(e) Stress/strain curves (and envelopes of scatter about the mean) of soil treated with 10% lime for the three curing methods. (see Fig. A.2.3(a)).

APPENDIX 3

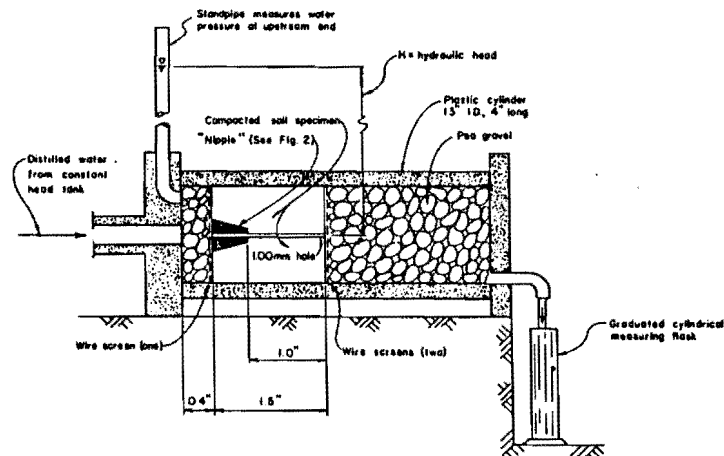
PINHOLE EROSION TEST

A 3.1. Introduction

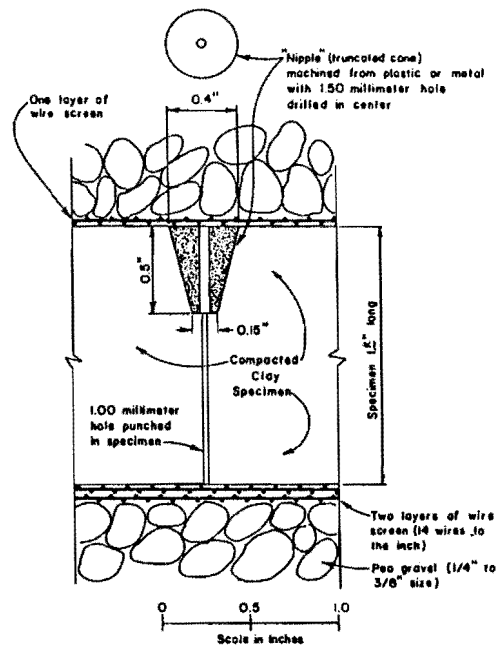
The pinhole erodibility test was first proposed by Sherard et. al. (1976). The test was developed to determine the dispersibility of clay soils used in earth dams. Evans (1977) modified the apparatus to specifically test in-situ samples of loessial soils from Banks Peninsula. This modified version is used in this study.

A 3.2. Test procedure

- 1) An undisturbed tube sample (35mm diameter) of soil is obtained in the field and is kept at in-situ moisture content until testing is performed. In this study the 35mm tube sample was obtained from a recompact Proctor mould sample.
- 2) The tube sample is trimmed to a standard length of 50mm and a truncated conical hole is drilled in the centre at the top of the sample.(see fig A.3.1).
- 3) A 1mm diameter hole is drilled through the centre of the sample, from the bottom of the conical hole using a surgical needle.
- 4) The sample is then set up in the apparatus (see fig. A.3.1.) and water is passed through the pinhole via a 1.5mm diameter nipple hole under increasing heads of 50mm, 180mm, 380mm, and 1000mm.
- 5) Classification of the erosion resistance of the material is based on flow rates at each head (which is held for 10 minutes) and any visible sediment discharge. The flow rates are plotted on a time /flow graph (see figs. A.3.3) to indicate any major periods of "sustained" erosion within each head.



Pinhole Test Apparatus (1 in. = 25.4 mm)



Section through Pinhole Test Specimen (1 in. = 25.4 mm)

Fig. A.3.1 Pinhole Erodibility test apparatus.
(Sherard et al., 1976)

A 3.3. Classification

In the original classification of Sherard et. al. (1976), the term dispersion is redefined to mean colloidal erodibility. Samples are classified on cloudiness and time of settling of a water-sediment suspension from the tested sample. Samples are divided into six classes. The first two classes (D1 and D2) indicate dispersion, the other four indicate that erosion has taken place without colloidal dispersion and are designated non-dispersive (ND) grades 1 - 4 depending on the head at which erosion is initiated.

Evans (1977) and Evans and Bell (1981) suggested minor modifications to the Sherard et. al. classification. Both emphasize that the extent of erosion under the various heads as the principle criteria separating the classes. The degree of water cloudiness is less important. However, the D - ND terminology is retained.

The validity of such a classification has been questioned by Yetton (1986) and even Sherard et. al. (1976a) themselves. Research by Yetton (1986) at the University of Canterbury shows that erodibility as defined by Sherard et. al. did not correlate to dispersion as defined by Loveday and Pyle (1973) and Emerson (1967). Yetton also found that erodibility was inversely correlated to the percentage of clay in soils tested.

A 3.4. The Yetton classification

Yetton proposed a modified classification for the pinhole test which is outlined in detail in his thesis (1986). The classification discards the term dispersion and concentrates on determining erodibility, which is dependant on a number of properties of the soil; slaking, dispersion, cement dissolution in water, grain size, and fluid induced stresses. The classification is outlined below.

- 1) From the record of water volume over time (an example of this record is shown in fig. A.3.2.) the average flow rate (ml/sec) is calculated for each minute period.
- 2) A graph is prepared with flow rate (Q) on the Y axis and time (minutes) on the x axis. The 10 minute head changes are indicated by vertical lines, and the maximum flow rates

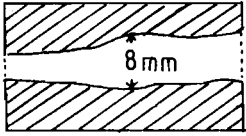
HEAD mm	TIME mins	FLOW Q ml ml/sec		SEDMT.			WATER				COMMENTS
				none	few	many	dark	pale	misty	clear	
50	00	00	0.00								<u>Sample</u> : CONTROL 1
	01	48	0.80		x			x			
	02	95	0.78		x			x			
	03	143	0.80		x				x		
	05	239	0.80		x				x		
	07	335	0.80		x				x		
	08	381	0.78		x				x		
	09	427	0.78		x				x		
	10	473	0.77		x				x		
180	01	573	1.67		x				x		
	02	770	1.62	x					x		
	03	768	1.63	x						x	
	05	963	1.63	x						x	
	07	1157	1.62	x						x	
	08	1254	1.62	x						x	
	09	1352	1.63	x						x	
	10	1450	1.63	x						x	
380	01	1598	2.47	x						x	
	02	1753	2.58		x				x		
	03	1923	2.83		x				x		
	05	2267	2.87		x				x		
	07	2620	2.94		x				x		
	08	2797	2.95		x				x		
	09	2975	2.97		x				x		
	10	3155	3.00		x				x		
1000	01	3434	4.65		x			x			Hole enlarged to 8mm ϕ in centre 
	02	3735	5.02		x			x			
	03	4048	5.22		x			x			
	05	4678	5.25		x			x			
	07	5322	5.37		x			x			
	08	5645	5.38		x			x			
	09	5965	5.33		x			x			
	10	6295	5.50		x			x			

Fig. A.3.2. Example of Pinhole Erodibility test sheet.
Control Sample.

possible without restriction by the sample are also shown.

- 3) The data is plotted and the points connected by straight lines. Graphs are given in figs. A.3. . Ignoring the first minute (2 minutes for the 1000 mm head) after each head change, the head at which "sustained erosion" first occurs for three or more minutes is noted. "Sustained erosion" is defined as that which produces a significant progressive increase in flow rate greater than 0.1 ml/sec. over a three minute period.
- 4) The head determined above is postscripted to E. eg. E 180. Non-erodible samples are classified NE.

Note:

- (a) If significant erosion first occurs at a low head, but only becomes sustained at the next higher head, intermediate erosion classes can be adopted. eg. E 380-1000.
- (b) The maximum possible flow rate for each head without sample restriction is required to ensure that any observed levelling off in increase in flow rate reflects sample characteristics and not the capacity of the equipment. This can be a particular problem with highly erodible material.

A 3.5 Results

The results of the pinhole erosion test are given in tables 3.1 and 3.3 and in the following time/flow graphs. (figs. A.3.). The results of erodibility are discussed in chapter Three, section 3.3.5.

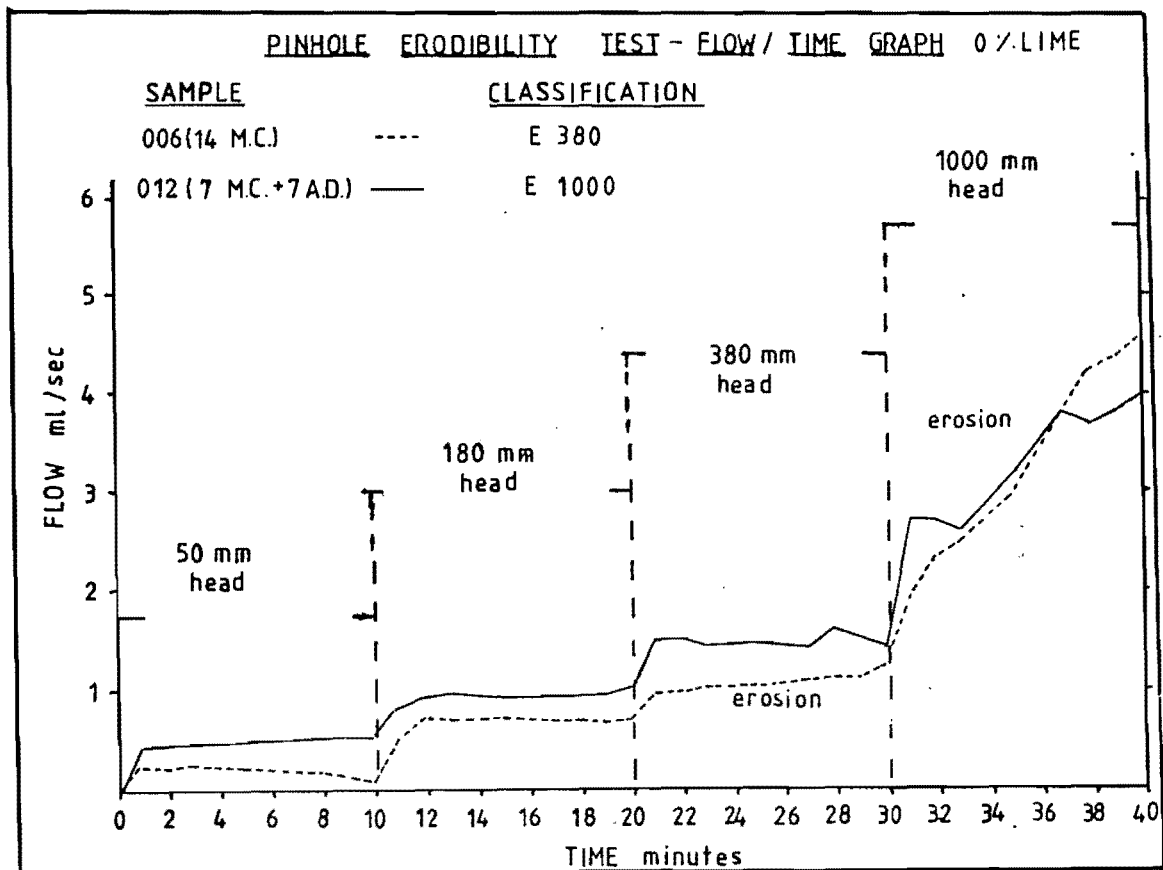
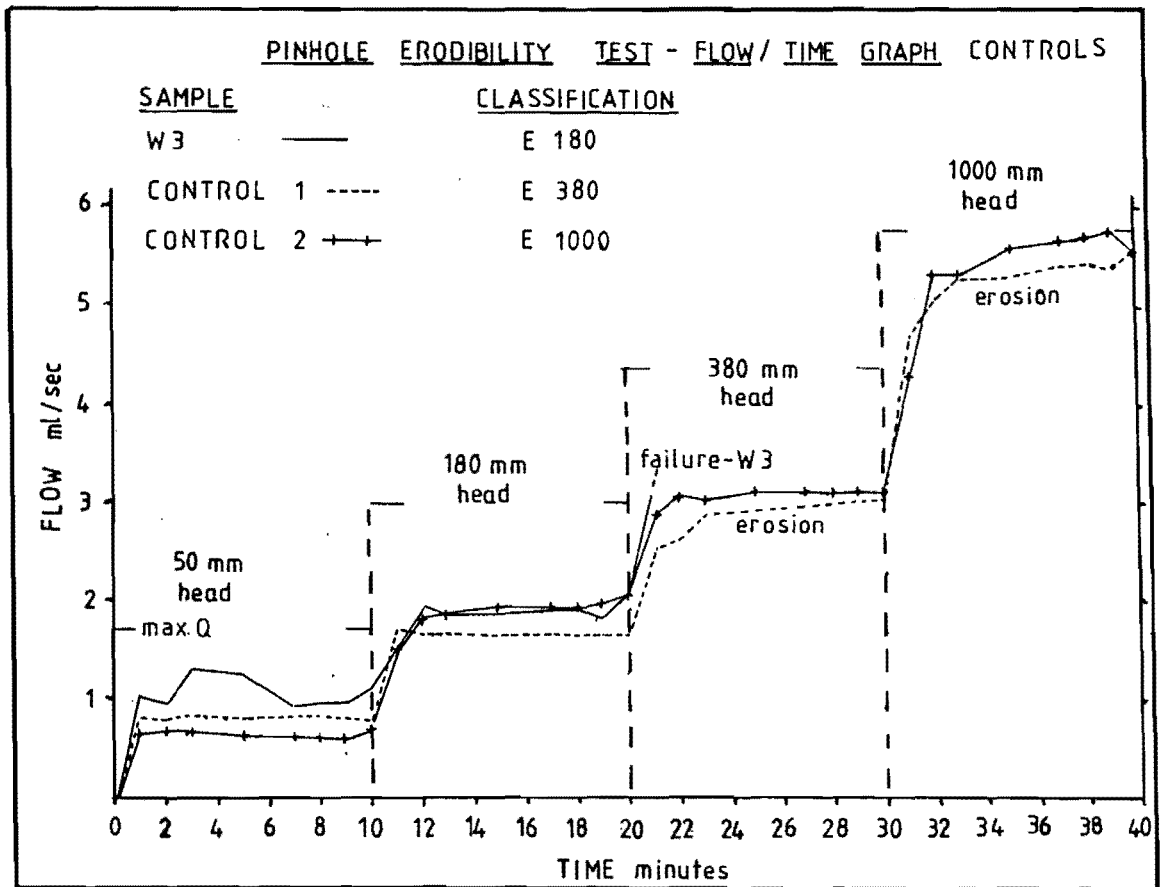


Fig. A.3.3(a) Pinhole erodibility flow/time graphs for the field sample (W3), controls and 0% lime.

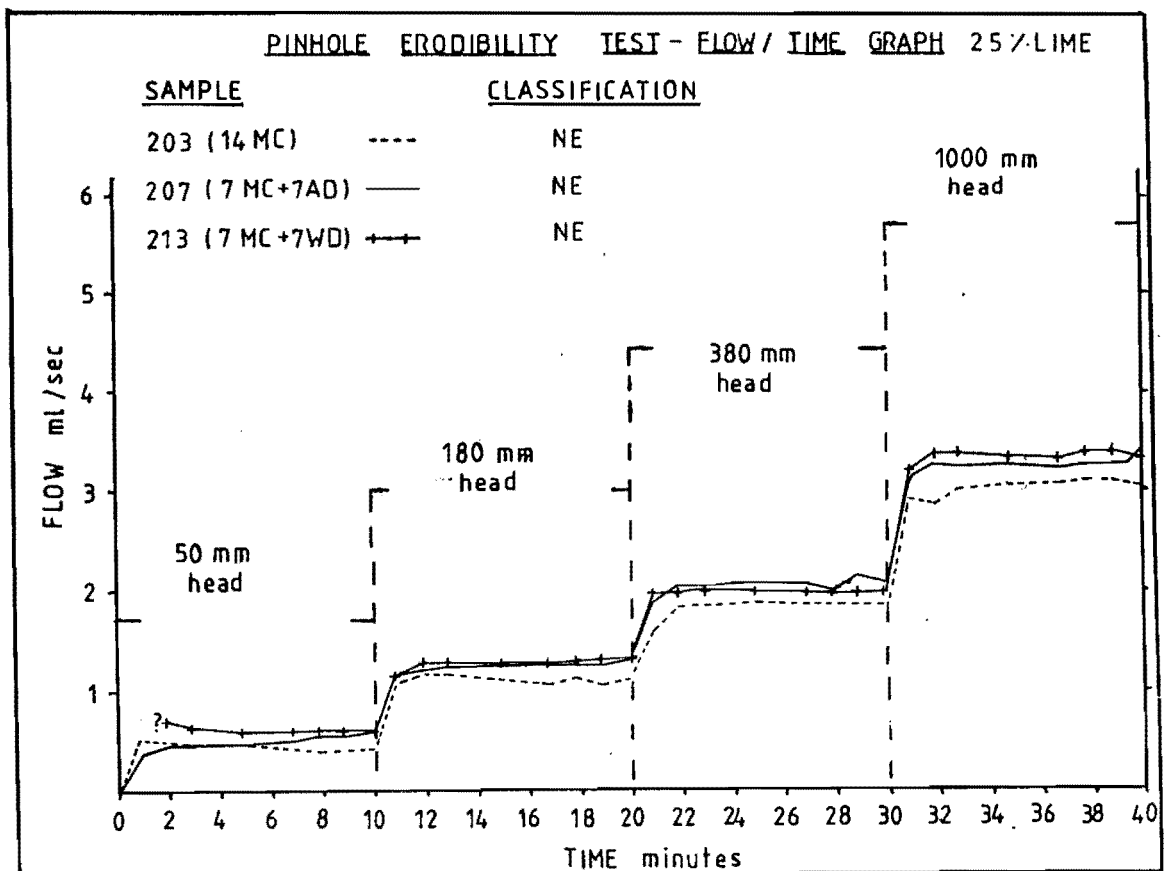
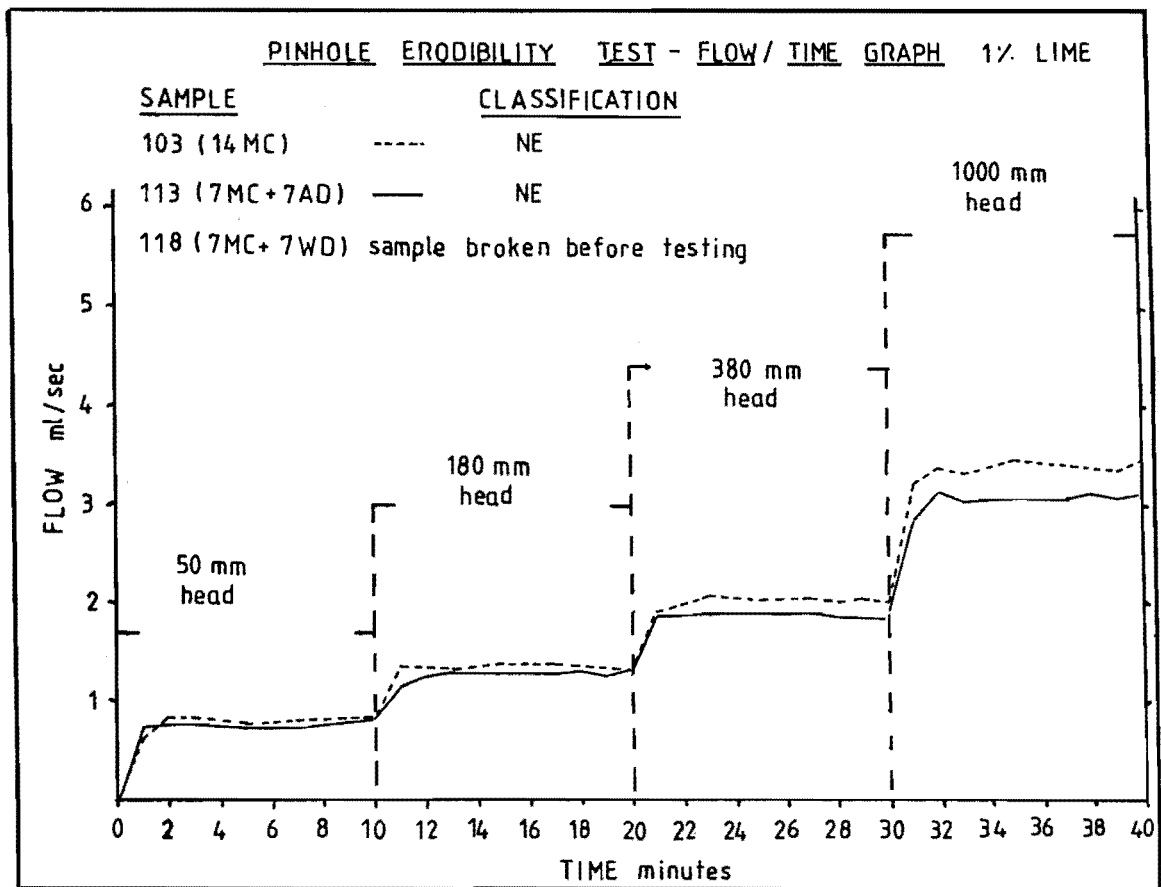


Fig. A.3.3(b) Pinhole erodibility flow/time graphs for 1% and 2.5% lime treated samples.

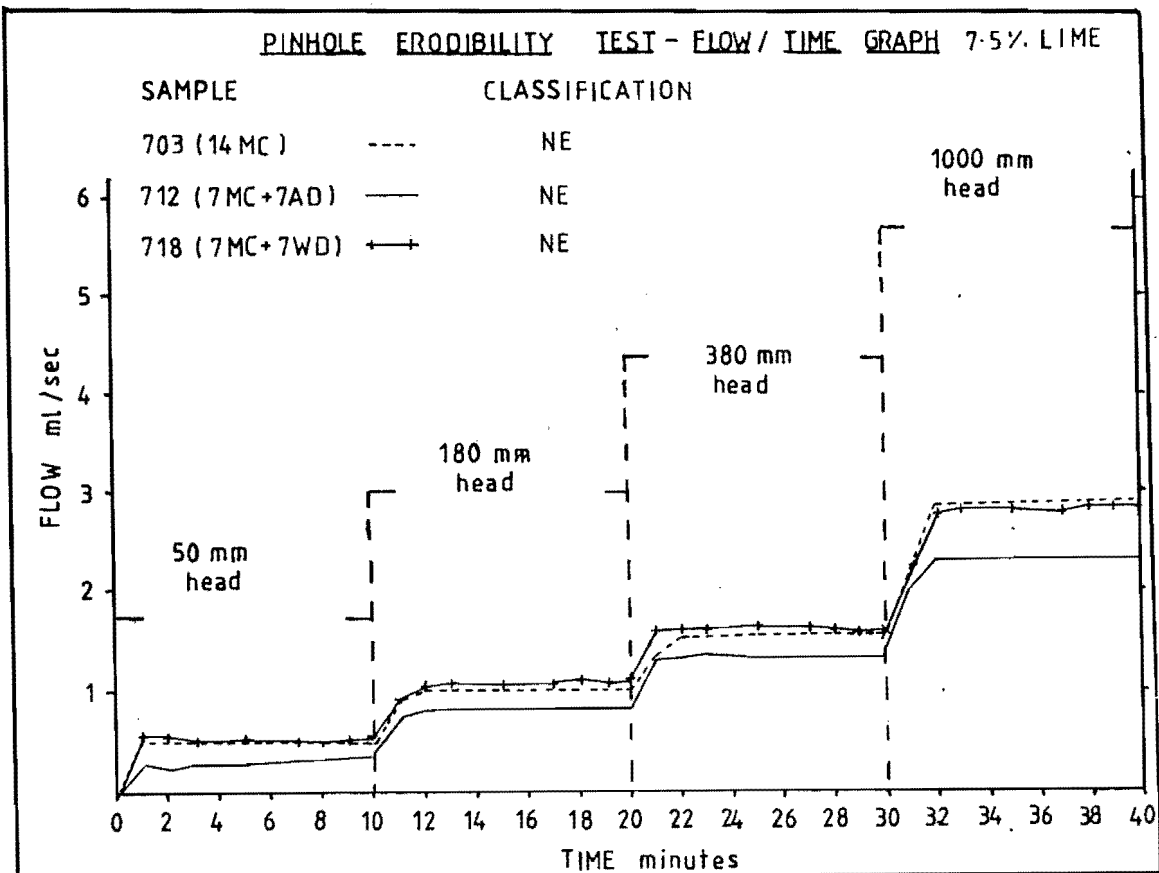
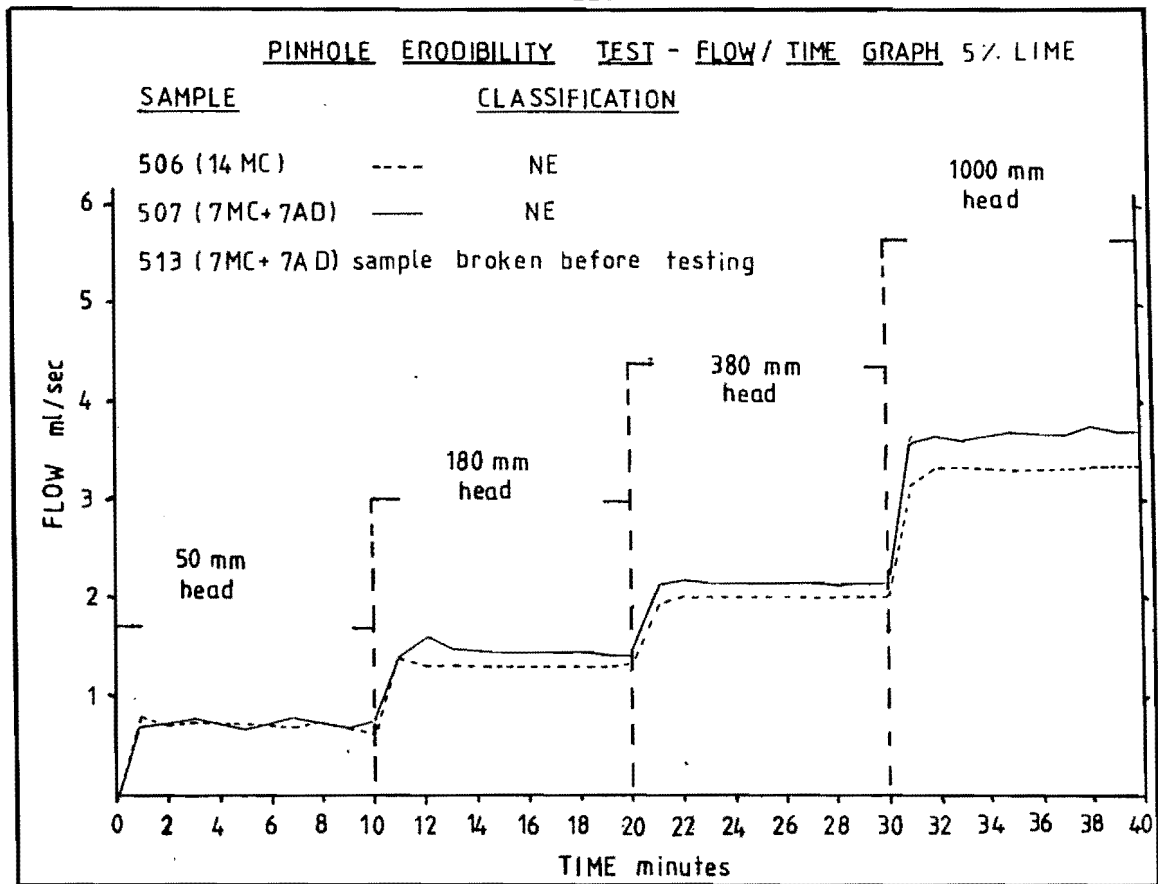


Fig. A.3.3(c) Pinhole erodibility flow/time graphs for 5% and 7.5% lime treated samples.

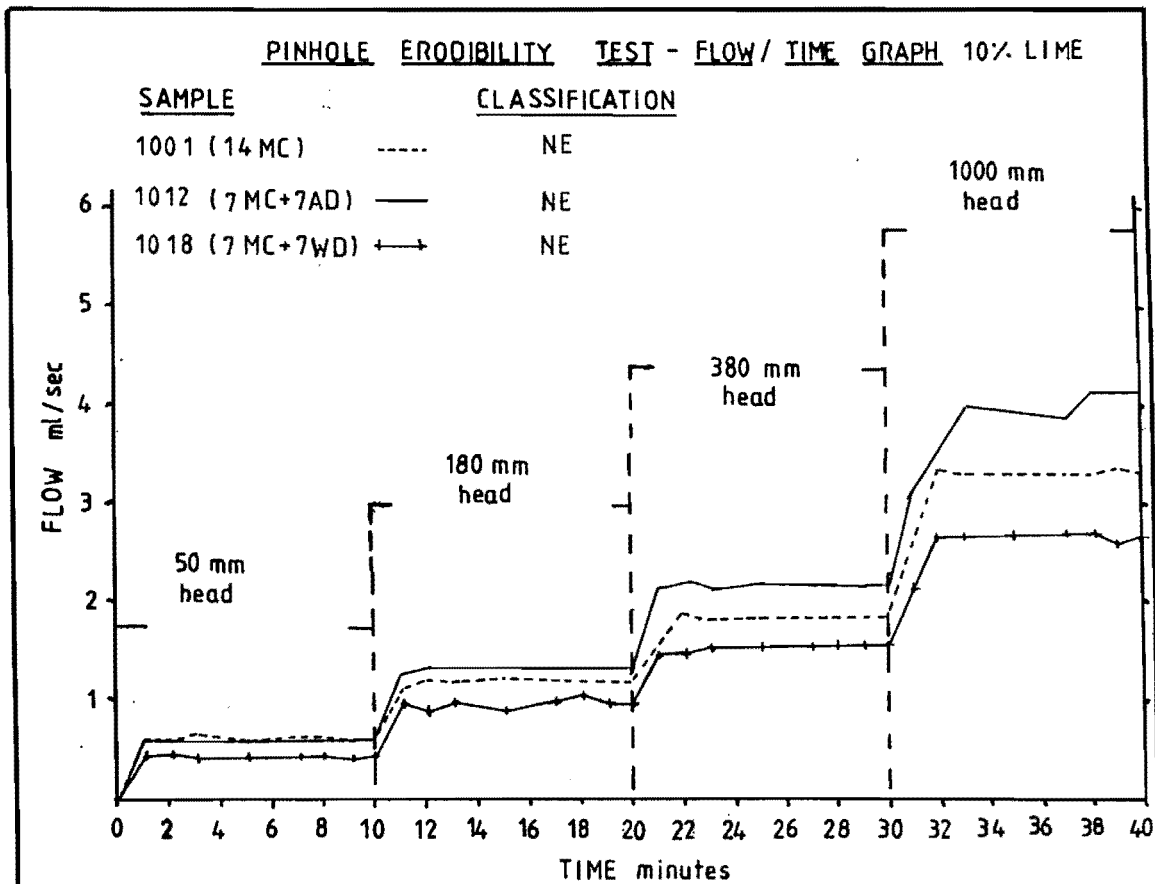


Fig. A.3.3(d) Pinhole erodibility flow/time graphs for 10% lime treated samples.

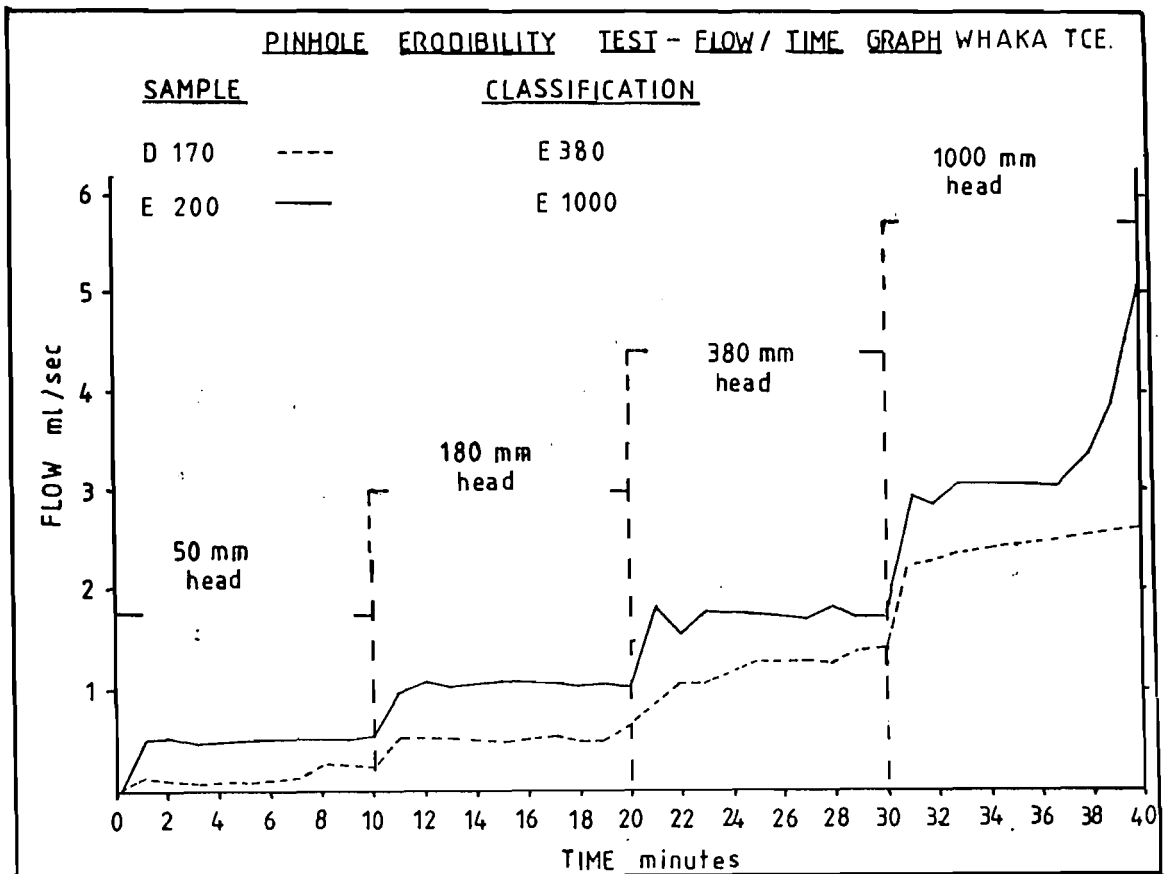
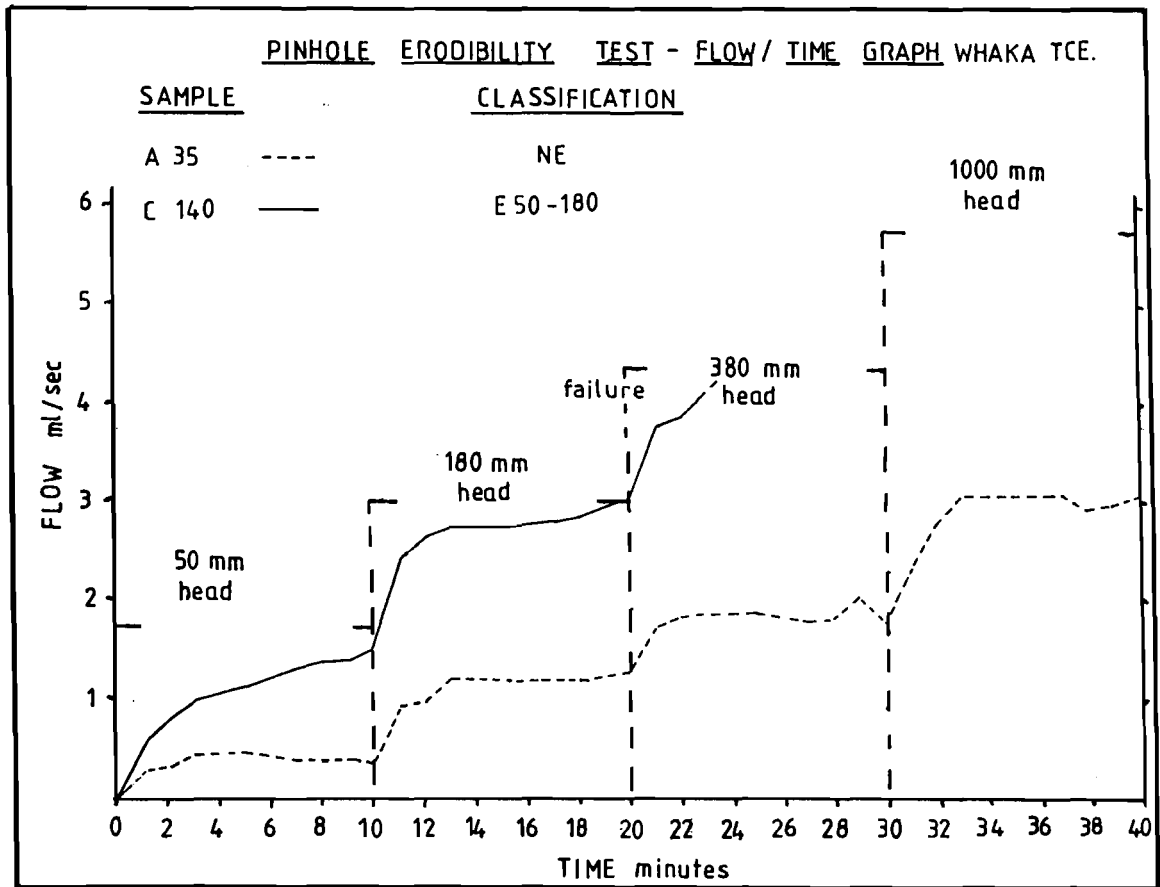


Fig. A.3.3(e) Pinhole erodibility flow/time graphs for samples from the tunnel gully erosion profile, Whaka Tce.

APPENDIX 4

EMERSON DISPERSION TEST

A 4.1 Introduction

The Emerson dispersion test (Emerson 1967) divides the soil aggregates into seven classes by observing the coherence of the clay fraction after reacting aggregates with water. The reactions carried out are;

- 1) immersion of dry aggregates in water,
- 2) immersion of wet remoulded aggregates in water, and
- 3) suspension of aggregates in water.

A 4.2. Procedure

- 1) An air dry soil aggregate (3-5 mm across) is dropped into 50ml of distilled water in a beaker. At 2 and 20 hours after immersion a visual judgement is made of the degree of dispersion.
- 2) If little or no dispersion has taken place after 20 hours, then the sample is remoulded to a water content approximately that of the liquid limit of the soil. Cubes (5mm x 5mm) are then immersed in water and observed for dispersion as in 1.
- 3) If no dispersion has taken place then the sample is tested for the presence of carbonate or gypsum. If these are absent, the soil is made up into a 1:5 aggregate-water suspension and visual observations made for dispersion or flocculation.

A 4.3 Classifying aggregates

The first separation of aggregates is made according to whether the dry aggregates slake when immersed in water (see fig. A.4.1). Most aggregates in fact slake due to the stresses imposed by trapped air and by swelling. These aggregates are placed in classes 1-6

depending on the degree of dispersion as outlined in the procedure. Those that do not slake are divided into two classes; class 8 aggregates are unchanged, whereas class 7 aggregates swell but remain coherent.

A 4.4. Results

Results for the Emerson dispersion test are given in tables 3.1 and 3.3, and are discussed in chapter Three, section 3.3.7.

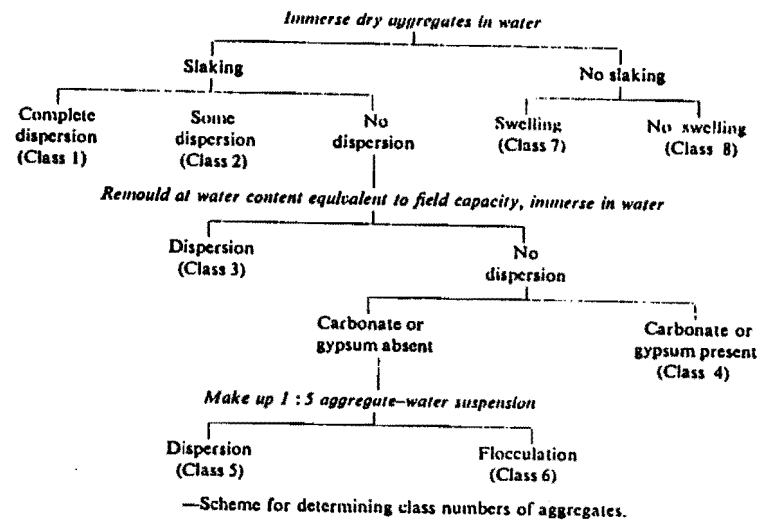


Fig. A.4.1. Emerson Dispersion test Classification.
(Emerson, 1967).

APPENDIX 5

GRAINSIZE ANALYSIS

A 5.1 Introduction

Grainsize analyses were carried out to determine the effects of lime addition on particle size distribution.

A 5.2 Procedure

Grainsize analyses of whole samples were carried out using techniques outlined in Lewis (1981). The sand fraction was determined by dry sieving and the silt and clay fraction by hydrometer analysis.

Grainsize was determined for the field sample (W3), the recompacted untreated soil, and the soil treated with 1%, 2.5%, 5%, 7.5%, and 10% lime. Samples were taken from the 14 day moist cured moulds.

Disaggregation prior to wet seiving was done by hand in a solution of distilled water and deflocculant (calgon) for a period of 10 minutes. For samples containing higher percentages of lime (5%, 7.5%, and 10%) additional calgon had to be added to prevent rapid flocculation.

A 5.3 Results

Results are shown in the particle size distribution graphs (figs. A.5.1 a-g). Percentages of sand, silt, and clay are given in tables 3.1 and 3.2, and the results are discussed in chapter Three, section 3.3.4.

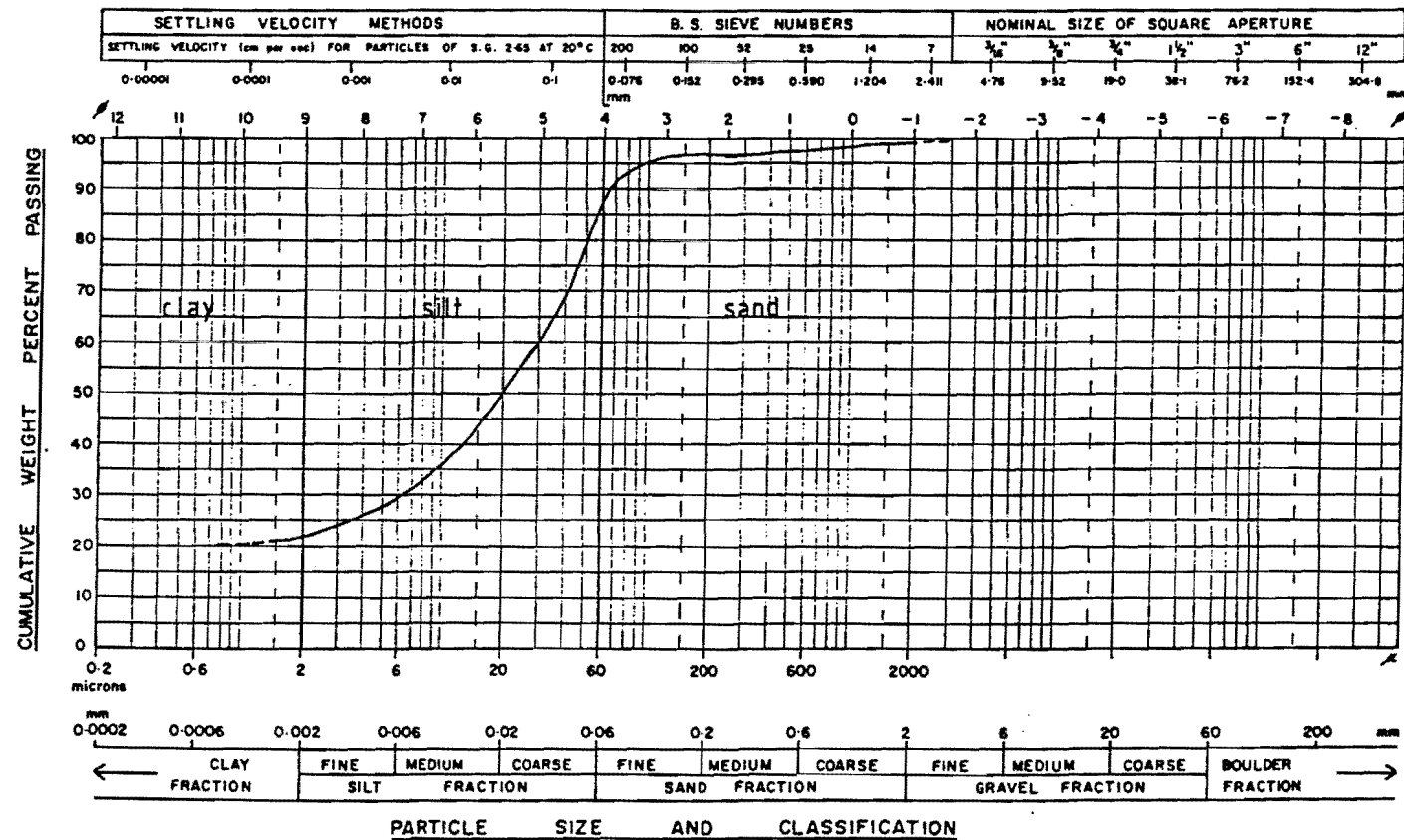
Fig. A.5.1(a) Grainsize distribution curve, field sample, Westmoreland.

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PARTICLE SIZE DISTRIBUTION — SEMI LOG PLOT

PROJECT ...WESTMORELAND..... SAMPLE NO ...W.3...field..... SAMPLED BY ANALYSED BY...SIEVE AND. HYDROMETER



PARTICLE SIZE DISTRIBUTION — SEMI LOG PLOT

PROJECT WESTMORELAND..... SAMPLE NO 006. 0% LIME..... SAMPLED BY ANALYSED BY SIEVE AND
HYDROMETER

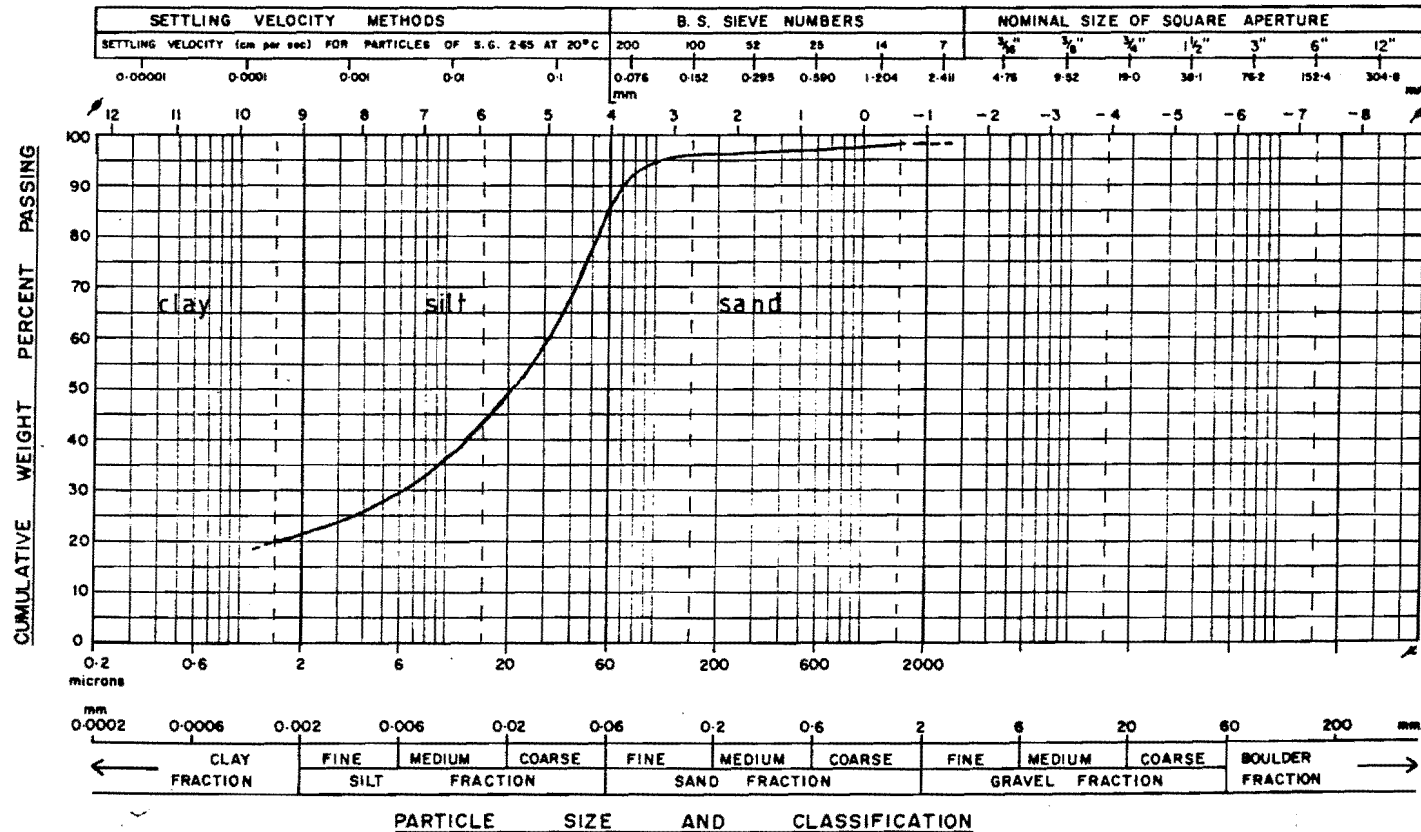


Fig. A.5.1 (b) Grainsize distribution curve, 0% (untreated) lime.

Fig. A.5.1(c) Grainsize distribution curve, 1 % lime.

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PARTICLE SIZE DISTRIBUTION — SEMI LOG PLOT

PROJECT WESTMORELAND..... SAMPLE NO 103..1%LIME..... SAMPLED BY ANALYSED BY SIEVE AND...
HYDROMETER

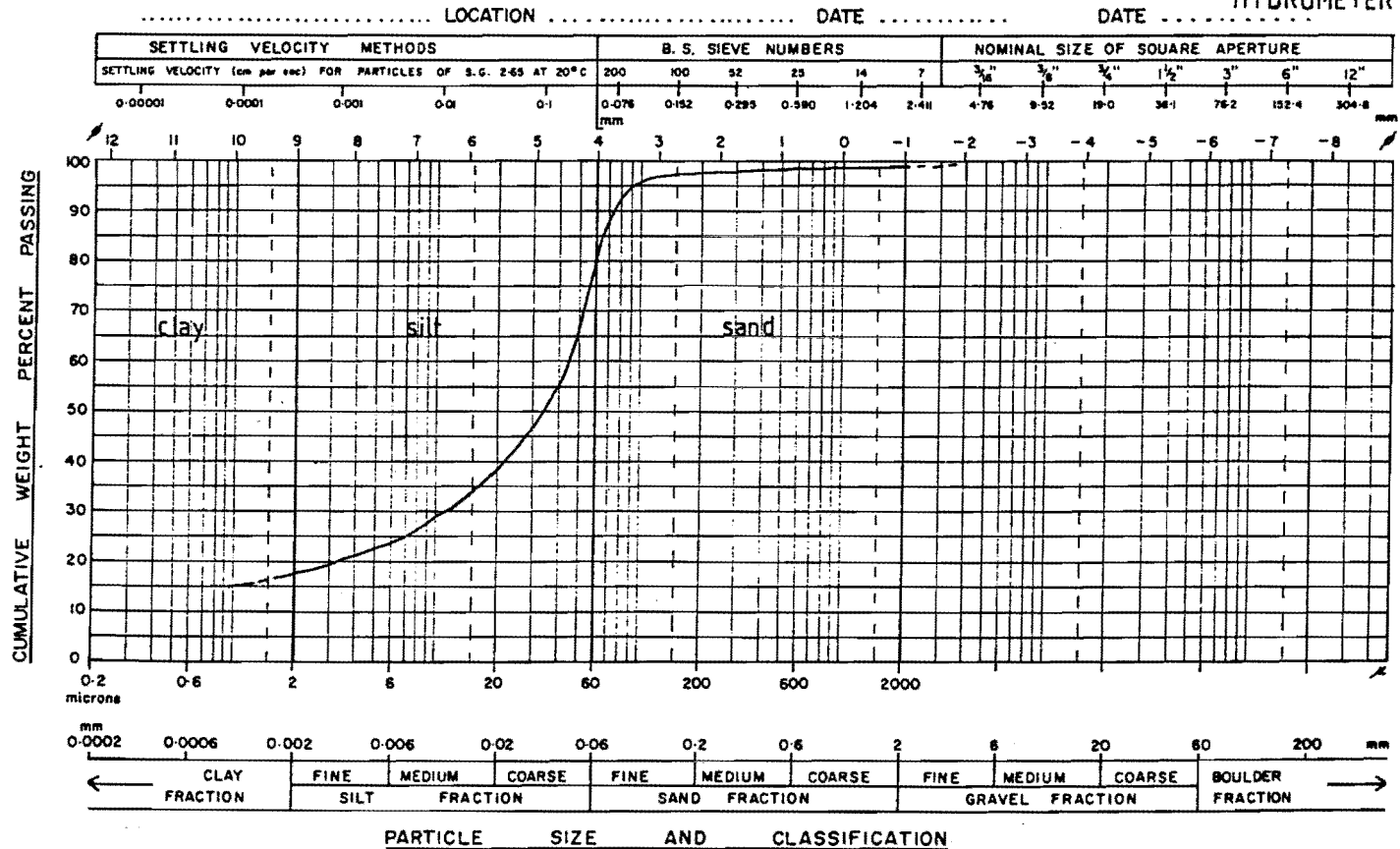


Fig. A.5.1(d) Grainsize distribution curve, 2.5% lime.

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PARTICLE SIZE DISTRIBUTION - SEMI LOG PLOT

PROJECT ..WEST.MORELAND..... SAMPLE NO 203. 2.5% LIME... SAMPLED BY ANALYSED BY SIEVE AND..
 LOCATION DATE DATE HYDROMETER

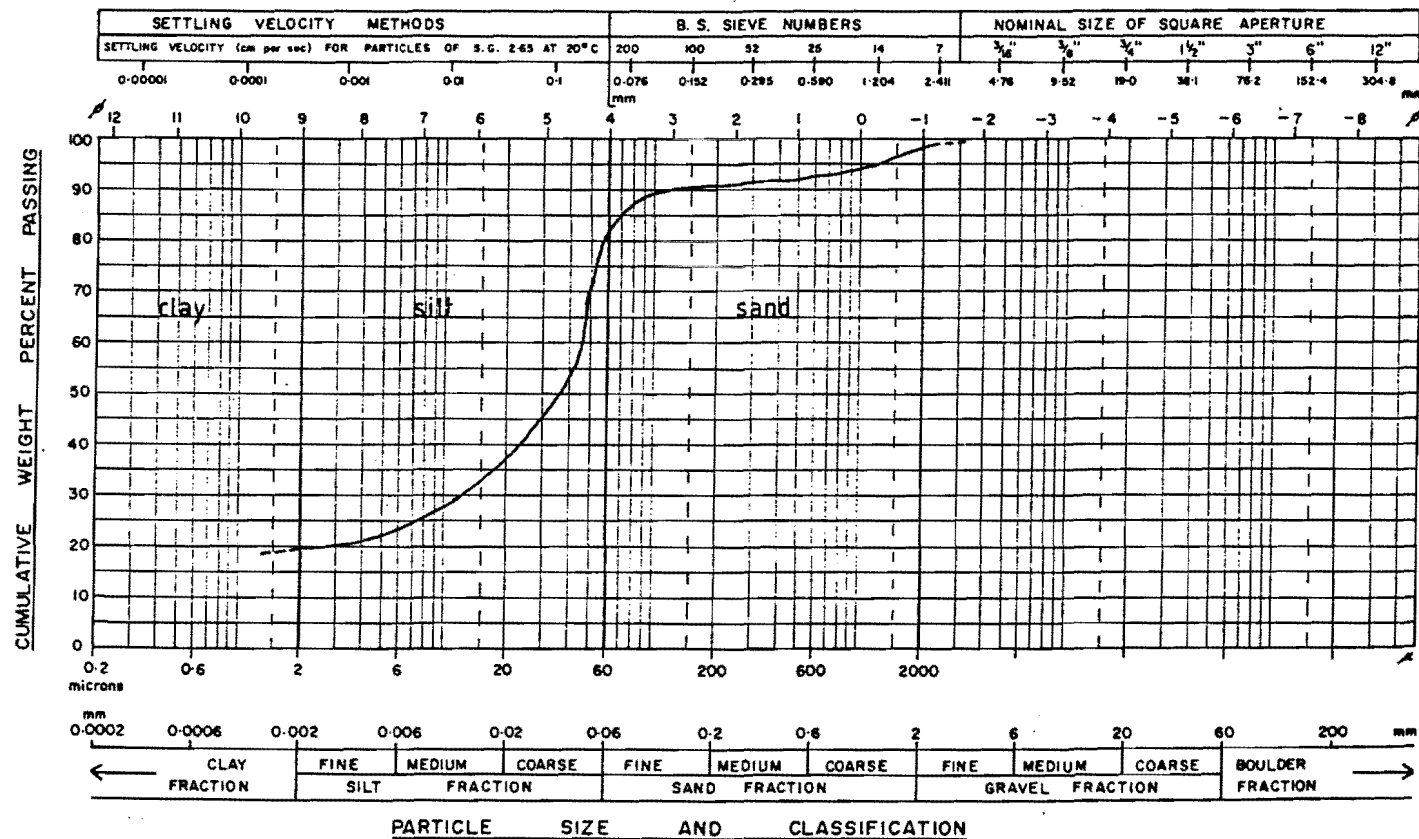


Fig. A.5.1(e) Grainsize distribution curve, 5% lime.

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PARTICLE SIZE DISTRIBUTION — SEMI LOG PLOT

PROJECT WESTMORELAND..... SAMPLE NO 506.5% LIME..... SAMPLED BY ANALYSED BY SJEVE AND...
 LOCATION DATE DATE HYDROMETER

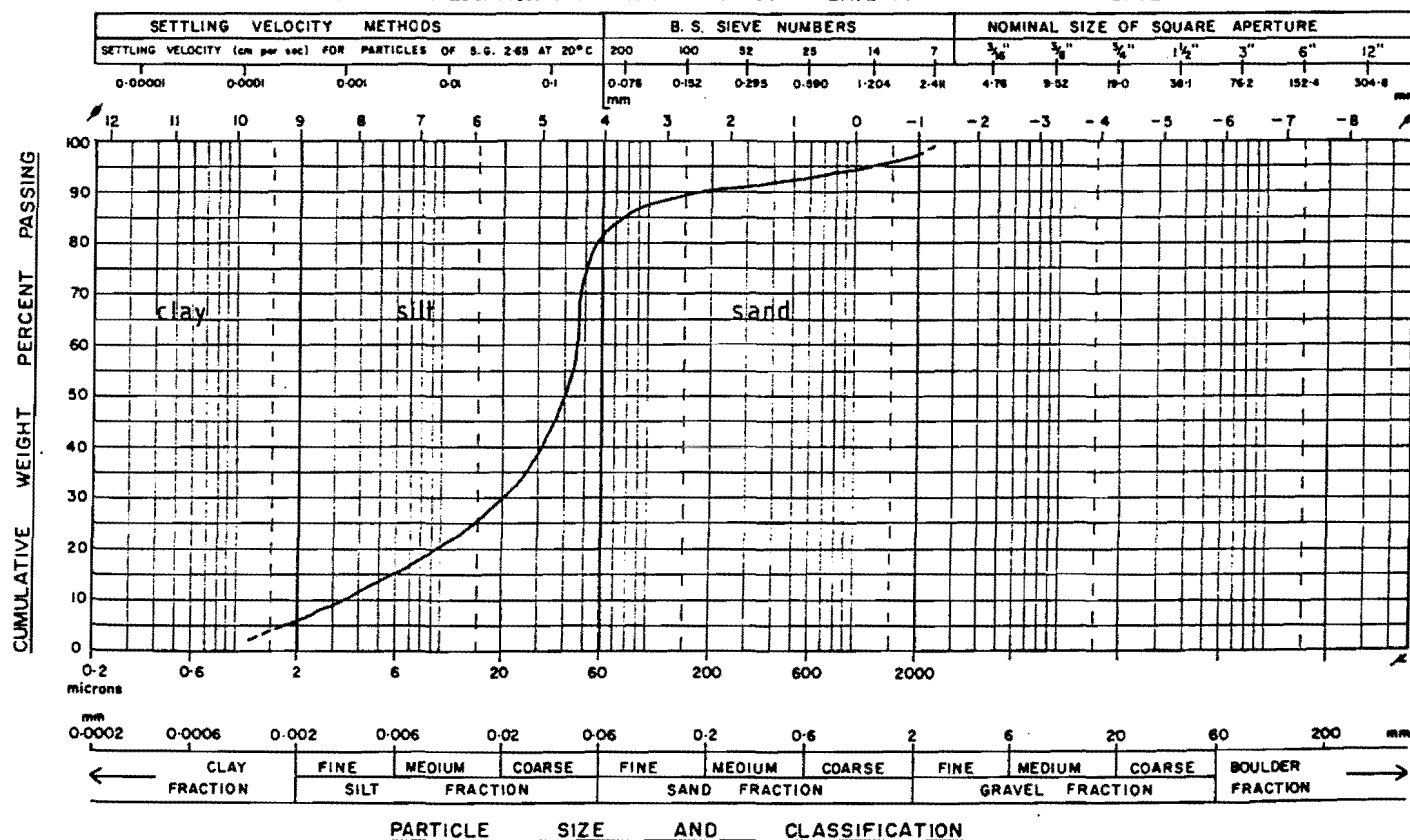


Fig. A.5.1(f) Grainsize distribution curve, 7.5% lime.

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PARTICLE SIZE DISTRIBUTION - SEMI LOG PLOT

PROJECT ...WESTMORELAND..... SAMPLE NO 703.7.5%LIME.... SAMPLED BY ANALYSED BY SJEVE AND. HYDROMETER

..... LOCATION DATE DATE

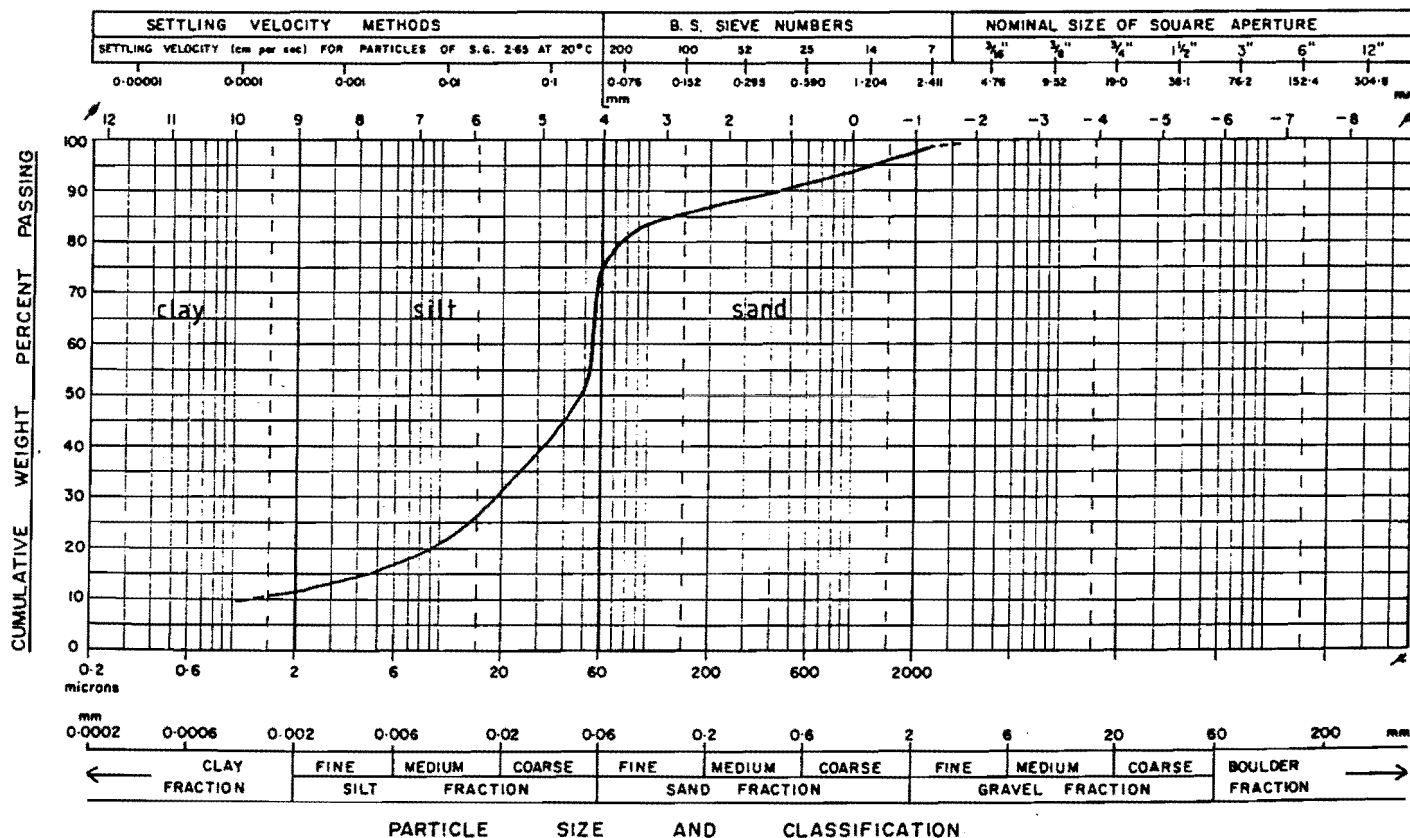


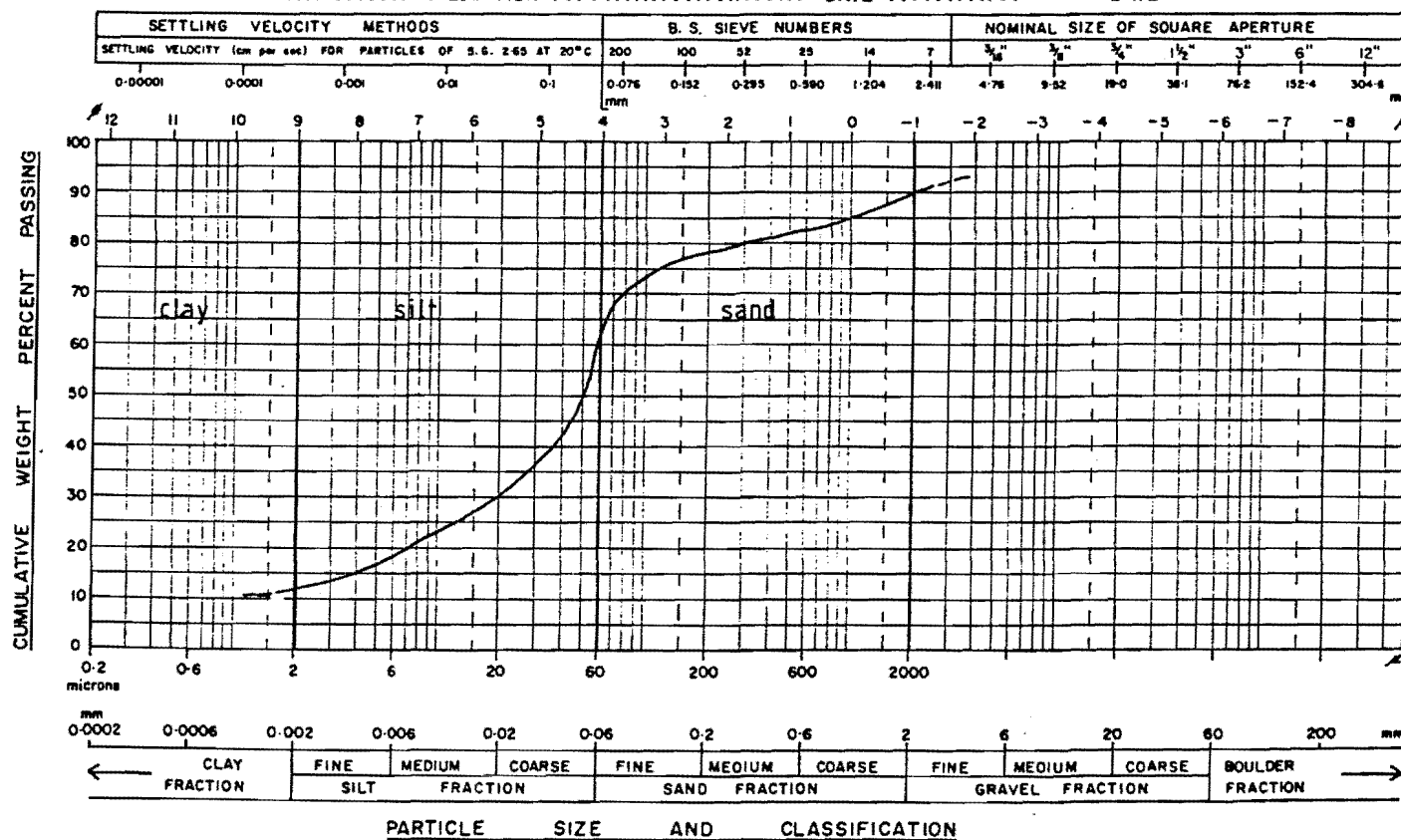
Fig. A.5.1(g) Grainsize distribution curve, 10% lime.

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PARTICLE SIZE DISTRIBUTION — SEMI LOG PLOT

PROJECT WESTMORELAND..... SAMPLE NO 1001..10% LIME.. SAMPLED BY ANALYSED BY SIEVE AND HYDROMETER
 LOCATION DATE DATE



APPENDIX 6

UNIAXIAL SWELLING STRAIN

A 6.1 General

The uniaxial swelling strain is by definition the change in (axial) length of a test specimen from the oven-dried state to the fully saturated state, expressed as the dimensionless ratio of change in length to original length. Swelling may be the result of either water uptake by clay minerals (with consequent lattice expansion), or of intergranular hydrostatic stresses exceeding cohesive strength.

A variation of the standard uniaxial swelling strain test is to determine the change in length from in-situ moisture content to full saturation: because of the dependence of potential swellability on moisture content, such a test gives a more realistic indication of any immediate swelling problem. In the testing of loessial samples, lateral confinement (using a stainless steel ring) is necessary to prevent collapse on immersion in water.

A 6.2 Procedure

Samples were obtained by driving a stainless steel confining ring into a compacted Proctor mould sample. The sample is then trimmed to the length of the confining ring. The length and diameter of the sample are noted. Samples were taken from moist cured moulds, air dried moulds and soaked moulds. The moisture content of the sample is determined prior to testing. Swelling strain is measured using a LVDT/chart recorder apparatus. (see fig. A.6.1.).

A 6.3. Calculations

The change in length is calculated from the chart recording. The uniaxial swelling strain is computed from the expression,

$$E_s = (dL / L) \times 100\%$$

where dL is the change in specimen length, and L is the original length.

A 6.4. Results

The results of the uniaxial swelling strain test are given in tables 3.1 and 3.3 and figures 3.4 and 3.5, and are discussed in chapter Three, section 3.3.8.

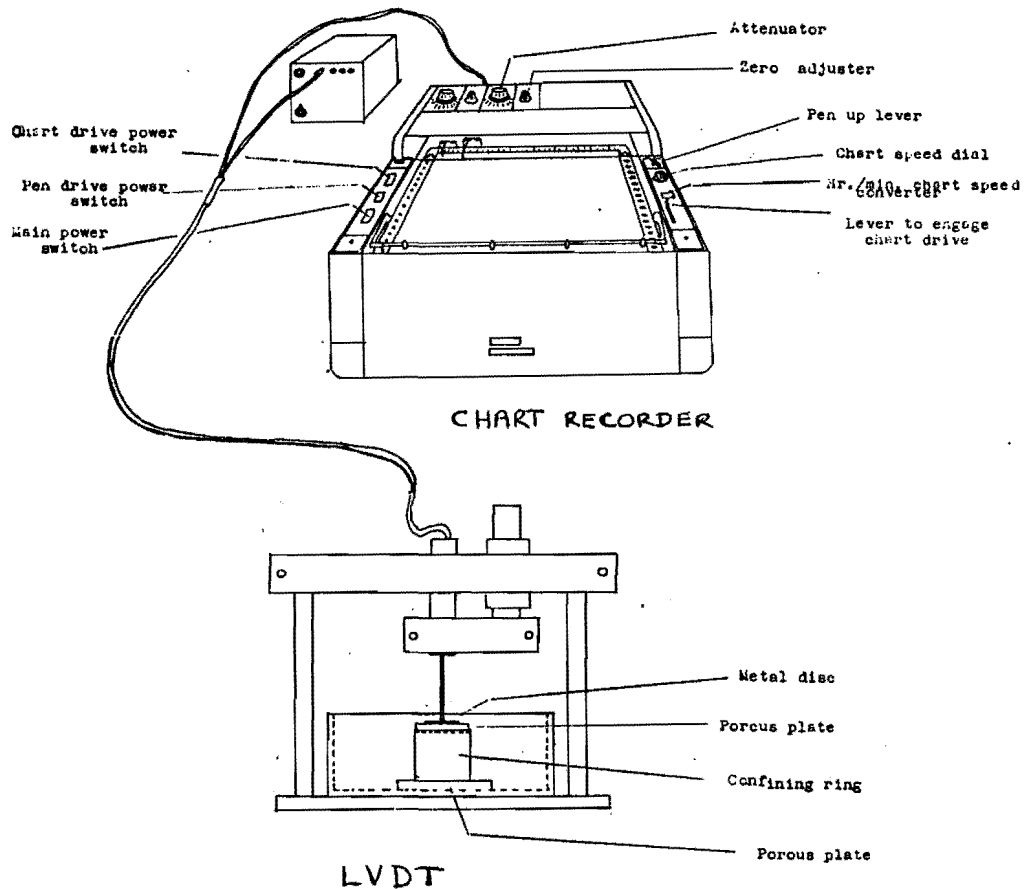


Fig. A.6.1. Swelling Strain measuring apparatus.

APPENDIX 7

CLAY MINERAL IDENTIFICATION

A 7.1. Introduction

Clay minerals of the untreated soil from Westmoreland were identified using X-ray diffraction analysis of an orientated mount of the clay fraction. The diffraction patterns of many clay minerals overlap, so the samples are solvated with glycerol (an organic compound) and heated in order to distinguish between the various clay minerals.

A 7.2. Sample preparation

Orientated slide mounts are prepared by removing a sample of 90 and finer ($< 2\mu\text{m}$) material from a settling column after grainsize analysis. The suspension is allowed to stand for several days before a few drops of the clay suspension are placed on a glass slide, and allowed to dry at room temperature.

A 7.3. Clay Mineral Identification

A Phillips X-ray diffractometer is used to identify the clay minerals. The following procedure is used to distinguish clay minerals.

- 1) The untreated slide is passed through the X-ray machine and a diffractogram is obtained from $2^\circ - 30^\circ 2\theta$, which enables detection of the composite clay minerals.
- 2) The slide is treated with a fine spray of glycerol and left to dry. The sample is rerun through the diffractometer from $2^\circ - 14^\circ 2\theta$. Solvation with glycerol increases the basal spacing of smectites from approximately 12-15 Å to 18 Å, enabling distinction from non swelling clays with similar peaks.
- 3) The sample is heated at 550°C for 2 hours. This

distinguishes the two peaks of chlorite (7\AA and 14\AA) from other minerals.

A 7.4 Results

The diffractogram given in figure A.7.1 is a representative of 3 samples of the untreated soil from Westmoreland subdivision. The diffractogram indicates that the soil contains mixed layered clays (illites), smectites, and possibly some vermiculite.

KEY

I ILLITE
 S SMECTITE
 V VERMICULITE ?
 QTZ. QUARTZ

Peak references in Ångströms

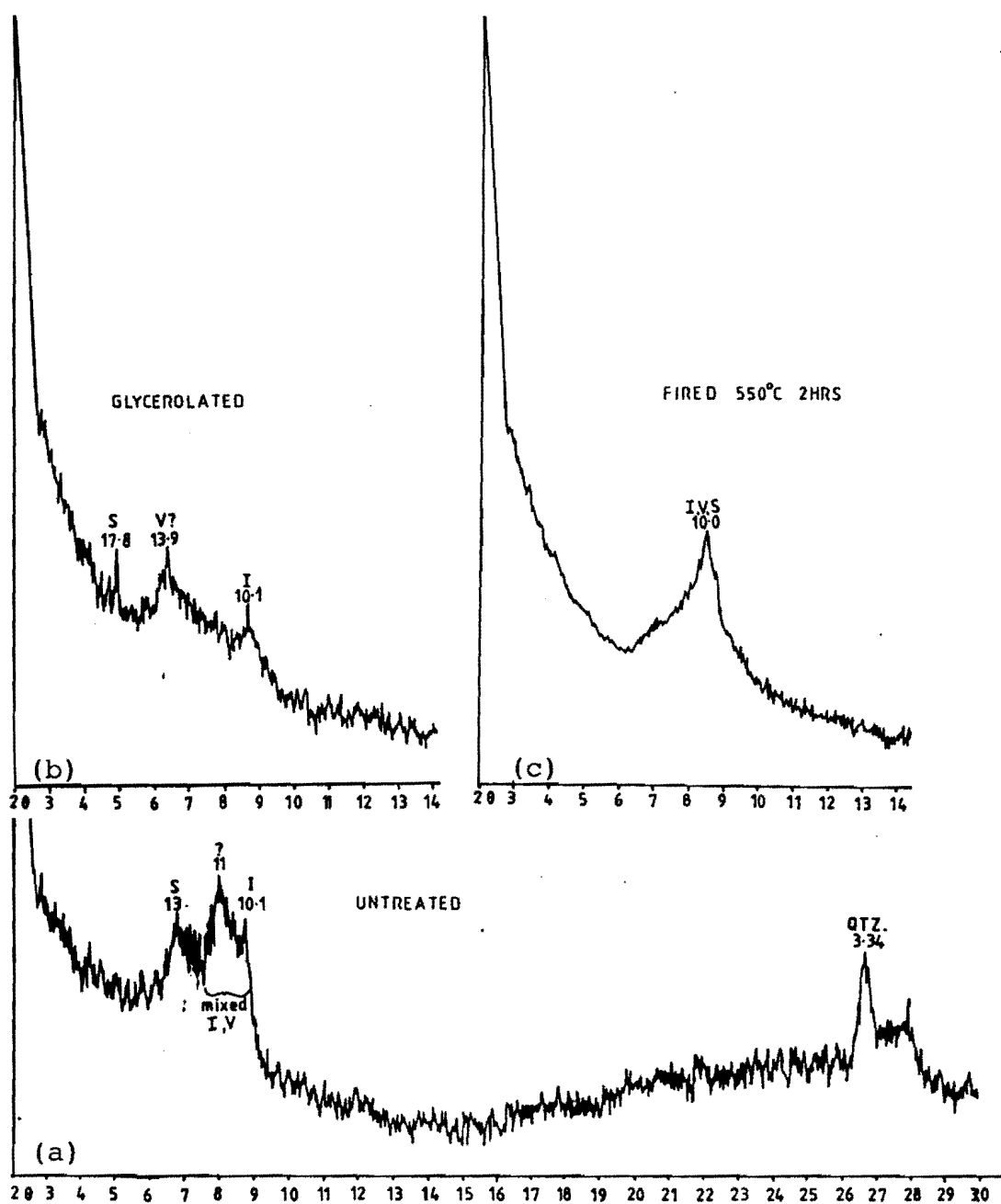


Fig. A.7.1 X-ray diffractogram of the untreated soil from Westmoreland. (a) untreated slide, (b) glycerolated slide, (c) fired slide.

APPENDIX 8

MISCELLANEOUS SOIL TESTS

A 8.1 Dry density / Water content relationship

NZS 4402 Part 2 (1981) Test 14, p. 16-19.

Modification: the same 2.5 kg of sample was used for each test.

A 8.2 Determination of the moisture content

NZS 4402 Part 1 (1980) Test 1, pp. 15-17.

A 8.3 Determination of the Atterberg limits

Liquid limit : NZS 4402 Part 1 (1980) Test 2, pp. 18-23.

Plastic limit : NZS 4402 Part 1 (1980) Test 3, pp. 24-26.

Plasticity index : NZS 4402 Part 1 (1980) Test 4, p. 27.

A 8.4 Determination of in-situ density

NZS 4402 Part 2 (1981) Test 17 (c), pp 40-42